



Assessment of Fire-Induced Damage and CFRP Retrofitting Strategies for GFRP-Reinforced Concrete Columns

A thesis

submitted to the Faculty of Graduate Studies
in partial fulfillment of the requirements for the

Degree of Master of Science

in

Civil Engineering
Lakehead University

by

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ABSTRACT

Fibre-reinforced polymers (FRPs) are increasingly used to improve the durability and sustainability of reinforced concrete structures. Glass FRP (GFRP) bars offer a corrosion-resistant alternative to steel reinforcement, making them attractive for harsh environments and long-term infrastructure. However, their performance under elevated temperatures remains a concern and has not been clearly investigated. GFRP-reinforced concrete (RC) members are vulnerable to thermal degradation, and the structural consequences of fire exposure (such as bond loss, strength reduction, and instability) are not yet fully understood. One promising repair method is external confinement using carbon FRP (CFRP) wraps, which provide high strength and can be efficiently applied in the field. This study investigates the use of CFRP jacketing to restore axial capacity in fire-damaged GFRP-RC columns. A two-phase experimental program was conducted on fourteen square columns (200×200 mm), including both short (1000 mm) and slender (1500 mm) specimens, with variations in fire exposure (one or two hours) and confinement level (one or two CFRP layers). Ambient specimens had strain gauges surface-mounted on the longitudinal GFRP bars prior to casting, while fire-tested

Specimens were embedded with thermocouples to monitor internal temperatures during exposure. Nine columns were fire-tested in accordance with the CAN/ULC-S101 standard, and seven were subsequently retrofitted with CFRP. All specimens were tested under axial compression with pin-ended boundary conditions. Lateral displacement was measured using LVDTs positioned at mid-height in both orthogonal directions. The program evaluated how fire duration, slenderness, and confinement affect residual and regained axial capacity. Test results showed that fire exposure caused significant capacity loss, with reductions amplified by fire duration and slenderness. Two-hour specimens retained only a fraction of their original strength,

and slender columns showed greater thermal penetration and instability. Internal temperatures exceeded critical thresholds for GFRP degradation, leading to visible damage in bars and concrete. CFRP jacketing enhanced strength and ductility in all retrofitted columns. Confinement became more effective with increasing fire damage, as retrofitted columns showed greater strength gains relative to their unwrapped counterparts. However, even with multiple layers, severely damaged columns remained well below ambient strength. These results highlight both the potential and limitations of CFRP confinement in post-fire rehabilitation of GFRP-RC compression members.

KEYWORDS

Glass fibre-reinforced polymer (GFRP), Carbon fibre-reinforced polymer (CFRP), Reinforced concrete, Fire Safety Engineering, Residual axial capacity, Regained axial capacity, Post-fire rehabilitation, FRP confinement, Thermal degradation, Fire-damaged columns, CFRP jacketing, CAN/ULC-S101, Slenderness effects, Ductility enhancement.

CO-AUTHORSHIP STATMENT

This thesis has been prepared in an Integrated-Article format. As such, the entirety of Sections 2 and 3 has either been published or is currently under review for journal or conference publication. These sections were co-authored as part of the publication process with Dr. Ahmed Elshaer (A.E.) and Dr. Ahmed Bediwy (A.B.), both part of Lakehead University's Department of Civil Engineering. Their contributions are identified using the Contributor Roles Taxonomy (CRediT) outlined below. Minor editorial changes have been made to ensure consistency and coherence within the thesis format. Information from external sources has been cited below.

Chapter 2: Experimental Investigation of Residual Strength in GFRP-RC Columns Subjected to Fire

McTavish K, Bediwy A, Elshaer A. GFRP-Reinforced Concrete Column Capacity Reduction due to Fire Exposure. CSCE-2025 Structures Specialty Conference, Winnipeg, MB: May 28-30, 2025.

McTavish, K., Elshaer, and A. Bediwy, A. Experimental Investigation of Residual Strength in GFRP-RC Columns Subjected to Fire. (to be submitted to a journal publication)

CRediT Taxonomy:

Conceptualization, K.M., A.E., A.B.; investigation, K.M.; methodology, K.M., A.B.; formal analysis, K.M.; supervision, A.E., A.B.; writing—original draft preparation, K.M.; writing—review and editing, A.E., A.B.

Chapter 3: Behaviour of Fire-Damaged GFRP-Reinforced Concrete Columns Retrofitted with CFRP Wraps.

McTavish, K., Bediwy, A., and Elshaer, A. Behaviour of Fire-Damaged GFRP-Reinforced Concrete Columns Retrofitted with CFRP Wraps. (to be submitted to a journal publication)

CRedit Taxonomy:

Conceptualization, K.M., A.E., A.B.; investigation, K.M.; methodology, K.M., A.B.; formal analysis, K.M.; supervision, A.E., A.B.; writing—original draft preparation, K.M.; writing—review and editing, A.E., A.B.

ACKNOWLEDGEMENTS

I would like to express my sincere appreciation to my supervisors, Dr. Ahmed Elshaer and Dr. Ahmed Bediwy, for the opportunity to pursue graduate research under their supervision. Their academic input helped guide the technical direction of the research, and I value the trust they placed in me to see it through. Completing this work has been a formative experience, and I am thankful for the opportunity to have contributed to this area of research.

I would also like to thank Dr. Mohammed El-Gendy (Lakehead University) and Dr. Ramla Qureshi (McMaster University) for their time and willingness to serve on my committee. I appreciate their involvement in the final stages of this degree.

This research would not have been possible without the support of the laboratory staff in the Civil Engineering Department at Lakehead University. Their technical knowledge and assistance were instrumental during specimen preparation and testing. I would also like to acknowledge LUFTRL for access to the testing facility and equipment, and thank Dr. Salem for the technical support during the fire testing.

Financial support for this work was provided through the Ontario Graduate Scholarship (OGS), along with additional awards and bursaries granted through Lakehead University, the Faculty of Graduate Studies, the Department of Civil Engineering, and my supervisors. This funding allowed me to focus on my research and complete this degree.

I am thankful to the many students, colleagues, and friends who offered encouragement, collaboration, and practical help throughout this journey. Their presence played a meaningful role in keeping me sane through the more challenging moments of this process. Finally, I am grateful to my family for their ongoing patience and understanding, and to my dog, Newton, whose joyful presence served as a daily reminder that there is more to life than research.

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LIST OF SYMBOLS

A_g	Gross Area of Concrete Specimen
A_{GFRP}	Total Area of Longitudinal GFRP Reinforcement
b	Column Width (Square Cross-Section)
D	Section Diagonal (Taken Equal to Column Width For Square Sections)
E_{CFRP}	Modulus of Elasticity of CFRP Material
f'_c	Specified Compressive Strength of Concrete (Cylinder Strength)
f_{CFRP}	Limiting CFRP Tensile Stress
f_{CFRPu}	Ultimate Tensile Strength of the CFRP Material
f_{co}	Unconfined Concrete Strength (Empirical Model by Shan et al., 2019)
f_{lcsa}	Confining Pressure using CSA S806 method
f_l	Confining Pressure using Empirical Method (Shan et al., 2019)
K	Effective Length Factor
k_c	Confinement Coefficient (CSA S806 Method)
k_e	Efficiency Factor (Empirical Model CSA S806 Method)
k_l	Confinement Parameter (CSA S806 Method)
L_e	Effective Length of Column
n_{CFRP}	Number of Layers of CFRP
P_{cc}	Confined axial capacity (CSA S806-based)
P_{em}	Confined axial capacity (empirical model by Shan et al., 2019)
P_{ex}	Experimental axial capacity
P_{cr}	Euler-based buckling capacity
P_{ro}	Nominal unconfined axial capacity for short columns (CSA S806-based)

P_r	Nominal unconfined axial capacity for slender columns (CSA S806-based)
P_{uc}	Theoretical unconfined axial capacity
r	Corner Radius of the Column Section
r_c	Normalized Corner Radius
t	Time (minutes)
t_{CFRP}	Thickness of one CFRP Layer
T	Furnace Temperature at time t
T_{cr}	Critical Temperature (50% strength loss)
T_d	Decomposition Temperature
T_g	Glass Transition Temperature
T_o	Initial Furnace Temperature (10°C)
T_s	Softening Temperature
α_1	Concrete Stress Block Factor
ϵ_{CFRP}	Design Hoop Strain in CFRP

LIST OF ABBREVIATIONS

ACI	American Concrete Institute
ASTM	American Society for Testing and Materials
CAN/ULC	Canadian Underwriters Laboratories of Canada
CFRP	Carbon Fibre-Reinforced Polymer
CSA	Canadian Standards Association
DAQ	Data Acquisition System
FRP	Fibre-Reinforced Polymer
GFRP	Glass Fibre-Reinforced Polymer
ISO	International Organization for Standardization
LUFTRL	Lakehead University Fire Testing and Research Laboratory
LVDT	Linear Variable Differential Transformer
NR	No Reinforcement
RC	Reinforced Concrete
SRC	Steel-Reinforced Concrete
TC	Thermocouple
UTM	Universal Testing Machine
VOA	Vertical Overlap Area

LIST OF SPECIMEN IDENTIFIER COMPONENTS

SH	Short Column
SL	Slender Column
F0	Ambient Specimen
F1	One-Hour Fire-Exposed Specimen
F2	Two-Hour Fire-Exposed Specimen
C0	Unwrapped Specimen
C1	One-Layer CFRP Wrapped Specimen
C2	Two-Layer CFRP Wrapped Specimen

Chapter 1 - Introduction

1.1 Background

Fire represents one of the most universal hazards to infrastructure, arising through diverse natural and human-driven causes such as lightning, electrical faults, industrial accidents, or secondary events like earthquakes rupturing gas lines. Unlike other natural disasters, which are often geographically restricted (for example, earthquakes along fault lines or flooding in low-lying regions), fires can occur in virtually any setting, from major urban settings to open natural landscapes. While fire risk is not new, its impacts are becoming increasingly urgent: the frequency and severity of wildfires alone are projected to rise by nearly 50% by the end of the century (UNEP, 2022). Beyond their immediate destructive capacity, fires compromise the stability of structural systems, interrupt essential services, and pose disproportionate risk to vertical load-bearing elements such as columns, where loss of capacity can initiate partial or global collapse. This universality and escalating threat underscore the importance of developing fire-resilient infrastructure across all material systems.

Concrete is one of the most widely used construction materials globally, not only for its strength, durability, and versatility, but also for its inherent non-combustibility. However, concrete's poor tensile performance necessitates the use of reinforcement in structural applications. For over a century, steel has been the standard reinforcing material, offering both high tensile strength and ductile behaviour. Yet, steel reinforcement is prone to corrosion, particularly in humid, saline, or de-icing salt environments (Ali et al., 2024). Despite mitigation techniques such as epoxy coatings and concrete alkalinity, corrosion remains a leading cause of premature deterioration in

reinforced concrete (RC) structures, accounting for 70–90% of failures in several field evaluations (Angst, 2018; Taffese & Nigussie, 2020).

Fibre-reinforced polymers (FRPs) have gained popularity as a corrosion-resistant alternative to steel (Elmessalami et al., 2019; Ganga et al., 2007). These composites consist of high-strength fibres—such as glass, carbon, basalt, or aramid—embedded in a polymer matrix (Mallick, 2007). They offer high tensile strength, are lightweight, non-corrosive, and can be manufactured as internal bars (e.g., GFRP) or external wraps (e.g., CFRP sheets) (Hawileh et al., 2016; Wang et al., 2007). In Canada, FRP use has continued to expand. For example, the Headingley (Taylor) Bridge near Winnipeg used GFRP bars and CFRP tendons and remains in service after almost three decades (Rizkalla et al., 1998). In Northwestern Ontario, the Nipigon River cable-stayed bridge utilizes GFRP in its precast deck panels, further supporting the region's move toward durable, corrosion-free infrastructure (Arafa et al., 2016).

Despite their advantages, FRP materials behave fundamentally differently from steel when used in reinforced concrete. FRPs are anisotropic and exhibit a linear-elastic response up to brittle rupture, without yielding or plastic deformation: this lack of ductility alters failure mechanisms and limits energy dissipation in FRP-reinforced members (Elmessalami et al., 2019; Jaitrong et al., 2025). In flexural members, this behaviour often necessitates conservative reinforcement ratios to avoid sudden bar rupture (Tran et al., 2025). In compression members, FRP's low transverse stiffness and limited compressive strength reduce its contribution to axial load resistance, and its influence is more often observed through stiffness effects, crack control, or confinement interaction (De Luca et al., 2010; Kobayashi, K., Fujisaki, 1995; Tobbi et al., 2014). Most design standards (e.g., CSA S806-12 (R2021); ACI PRC-440.2-23) assume no compressive strength contribution from GFRP bars in column design, resulting in conservative axial capacity estimates.

These differences in mechanical behaviour require alternative design philosophies and careful attention to how FRPs engage with concrete under load.

A critical limitation of FRP materials is their vulnerability to elevated temperatures (Bisby, Kodur, et al., 2005). The polymer matrix begins to degrade well below the temperatures that affect steel, leading to reductions in bond strength, stiffness, and overall structural capacity (S. Alsayed et al., 2012; Chowdhury et al., 2009). Threshold transition points, such as the glass transition (T_g), softening (T_s), and decomposition (T_d) temperatures govern the thermal behaviour of FRPs; exceeding these thresholds can lead to matrix softening, delamination, and resin decomposition, potentially reducing confinement or load-carrying ability (Bazli & Abolfazli, 2020; Fleischhaker et al., 2014; Robert & Benmokrane, 2010).

Additionally, while concrete is inherently non-combustible, it still undergoes significant deterioration at elevated temperatures. Strength loss is influenced by parameters such as aggregate type, concrete grade, and column geometry. Moisture evaporation, pore pressure build-up, and thermal incompatibility between the cement paste and aggregates contribute to cracking, microfissuring, and in severe cases, explosive spalling (Kodur & Dwaikat, 2010; Malik et al., 2021). Spalling has been attributed to both thermo-hygral processes, where vapour pressure accumulates in pores as moisture evaporates, and thermo-mechanical processes, where restrained thermal dilation near heated surfaces induces tensile stresses (Li et al., 2021; Liu et al., 2018). Research has shown that different aggregate mineralogies can influence residual strength, with siliceous concretes often degrading more rapidly than carbonate-based concretes due to mineral phase transformations and differences in thermal conductivity (Balaji et al., 2016; Malik et al., 2021). Geometric factors are also critical, as larger cross-sections provide greater thermal inertia that slows internal heating, while increased cover delays reinforcement heating and prolongs fire

resistance. For example, one study found that increasing the cover thickness from 40 mm to 60 mm could extend thermal resistance by approximately 80 minutes (Balaji et al., 2016). Column shape is equally relevant, as sharp corners promote localized spalling, whereas rounded or confined sections distribute stresses more evenly.

Another important factor is uneven heating: when fire exposure occurs from only one side of a column, a steep thermal gradient develops across the cross-section (Kodur & Raut, 2009). The hotter face expands more than the cooler side, inducing internal stresses and differential strain (Lie, 1992). This asymmetric expansion produces curvature and bending, effectively shifting the neutral axis and creating eccentric loading conditions even under concentric axial force. The resulting moment increases lateral deflections and can trigger instability or premature buckling, often before the material strength is fully exhausted (Alhadid & Youssef, 2016). Finally, higher sustained axial loads during fire exposure reduce stiffness, increase creep, and heighten the risk of premature failure (Bikhiet et al., 2014; Dotreppe et al., 1997). When combined with the relatively low thermal tolerance of FRP reinforcement, these vulnerabilities create a critical hazard in FRP-RC systems.

Despite this, current design standards, including CSA S806-12 (R2021) and ACI PRC-440.2-23, offer limited guidance on the performance of FRP-reinforced members during or after fire exposure (Hajiloo & Green, 2019). FRP materials are well-suited for use in remote or corrosive regions, where their lightweight, durability, and low-maintenance characteristics offer practical advantages (Fam & Son, 2008). However, many of these areas also face elevated wildfire risk and limited access to post-fire repair (Coogan et al., 2019). Addressing these knowledge gaps is critical to ensuring that the benefits of FRP can be confidently extended to high-risk or remote applications.

Externally bonded FRP wraps, particularly carbon FRP (CFRP), have been widely adopted for retrofitting and strengthening reinforced concrete columns (Bilotta et al., 2011; Pohoryles et al., 2019). These techniques provide lateral confinement, enhance axial capacity, and improve ductility by restricting transverse expansion and delaying instability (Chen et al., 2013). CFRP jacketing is especially valued for its high strength-to-weight ratio, ease of application, and minimal added bulk, making it suitable for in-place strengthening of aging or deficient structures (Farrokh Ghatte et al., 2019). While extensively studied under ambient conditions, the application of CFRP confinement in thermally degraded or fire-damaged members remains relatively underexplored.

To date, most fire testing of FRP-RC components has concentrated on beams and slabs, with limited attention given to compression members (Abd Elsalam et al., 2024). This is largely due to longstanding concerns over FRP's poor performance in compression and at elevated temperatures (Bazli & Abolfazli, 2020; Mohamed et al., 2014). However, as corrosion resistance continues to drive interest in non-metallic reinforcement, a clearer understanding of FRP-RC column behaviour under fire is becoming increasingly important. Additionally, while CFRP jacketing has shown promise for strengthening fire-damaged steel-RC columns (Bisby et al., 2011; Koppula et al., 2022), its effectiveness on FRP-RC columns subjected to fire remains an open question that warrants further investigation.

1.2 Research Problem

There is limited experimental research on the behaviour, degradation, and residual capacity loss of GFRP-RC columns under realistic fire exposure. Furthermore, limited standardized methods exist for evaluating or restoring the capacity of these members post-fire. While external confinement using CFRP wraps has proven effective for strengthening deteriorated or seismically deficient RC columns, its application to retrofitting fire-damaged GFRP-RC columns is largely untested. In the absence of reliable data on residual strength or retrofit performance, engineers are unable to confidently assess or rehabilitate these members after fire exposure. This knowledge gap limits the broader adoption of FRP in structural applications and constrains its use in fire-prone environments.

1.3 Objectives

The primary objective of this thesis is to experimentally evaluate the fire performance and post-fire rehabilitation of GFRP-reinforced concrete columns. Specifically, this work aims to:

- Investigate the residual axial capacity of GFRP-RC columns after exposure to one- and two-hour fire durations.
- Assess the influence of column slenderness on post-fire structural behaviour.
- Evaluate the effectiveness of CFRP confinement in restoring strength to fire-damaged GFRP-RC columns.
- Examine the combined effects of fire duration, confinement level (none, 1, or 2 CFRP layers), and column geometry on axial load performance.

By addressing these objectives, the research provides new data on the thermal vulnerability and rehabilitation potential of GFRP-RC columns, contributing to safer and more resilient FRP-based design.

1.4 Outline

This thesis is organized into four chapters:

- Chapter 1 provides background on FRP materials, fire vulnerability, and the research context, and defines the scope and objectives of the work.
- Chapter 2 presents *Journal Paper 1*, titled “*Experimental Investigation of Residual Strength in GFRP-RC Columns Subjected to Fire*,” which evaluates the post-fire axial capacity of fire-exposed GFRP-RC columns under concentric loading.
- Chapter 3 presents *Journal Paper 2*, titled “*Behaviour of Fire-Damaged GFRP-Reinforced Concrete Columns Retrofitted with CFRP Wraps*,” which investigates the effectiveness of external CFRP jacketing in restoring structural capacity.
- Chapter 4 summarizes the key findings of both studies, outlines limitations, and suggests future research directions related to fire performance and retrofitting of FRP-reinforced compression members.

Chapter 2 - Experimental Investigation of Residual Strength in GFRP-RC Columns Subjected to Fire

2.1 Introduction

Fibre-reinforced polymer (FRP) materials have been increasingly adopted in civil engineering due to their high tensile strength, corrosion resistance, and ease of installation. Among these, glass FRP (GFRP) has seen expanded use as internal reinforcement for concrete elements, particularly in flexural members. Long-term durability studies show that GFRP bars retain over 60% of their tensile strength after prolonged exposure to alkaline and chloride-rich environments, with tensile capacities often exceeding 1000 MPa (Lu et al., 2022). These properties have led to widespread use in bridges, marine structures, and parking garages, where durability and corrosion resistance are critical (Tarawneh & Majdalaweyh, 2020). However, its application in axially loaded columns remains controversial, despite the growing need for resilient infrastructure that reduces lifecycle maintenance. Current design codes, such as CSA S806-12 (R2021) and ACI 440.1 (R2023), either prohibit or severely limit the inclusion of FRP reinforcement in the axial capacity of concrete columns, reflecting longstanding concerns about the compressive behaviour of GFRP bars (Tu et al., 2019).

These concerns stem from early studies that raised doubts about the reliability of GFRP bars in compression due to their low stiffness, brittle failure mode, and poor confinement compatibility. (Kobayashi, K., Fujisaki, 1995) reported compressive strengths of only 30–40% of tensile capacity, while (S. H. Alsayed et al., 1999) observed a 13% reduction in axial capacity when GFRP was used instead of steel. Another study (De Luca et al., 2010) found that GFRP bars contributed less than 5% to axial capacity in rectangular columns with conventional tie spacing,

concluding that the contribution was too small to consider in design. Based on these findings, design standards assume that FRP reinforcement offers no meaningful contribution under compression and exclude it from both short and slender column design (American Concrete Institute (ACI), 2023; Canadian Standards Association (CSA), 2021; Choo et al., 2006). More recent research, however, has challenged this exclusion by demonstrating that GFRP bar contribution is highly dependent on confinement quality and slenderness ratio. Other studies (Tobbi et al., 2011; Tu et al., 2019) found that closer tie spacing and proper confinement detailing can enable the FRP bars to have a compressive contribution ranging from 3% to 10%. In circular columns with spiral reinforcement, studies have reported bar contributions up to 35%, depending on reinforcement ratio, spacing, and bar geometry (Afifi et al., 2014; Barua & El-Salakawy, 2020; Zeng et al., 2023). These findings suggest that both local confinement and global stability mechanisms play key roles in GFRP bar engagement (Hadhood et al., 2017).

Slenderness behaviour has also been recognized as a key factor influencing the axial performance of FRP-RC columns. Although Canadian codes (CSA 806, CSA A23.3) adopt a slenderness limit of 22 based on steel-reinforced design principles, studies have shown this limit may be overly permissive for GFRP-reinforced systems due to their lower stiffness and lack of ductility. Mirmiran et al. (2001) were the first to analytically study the slenderness behaviour of FRP-RC members and proposed a reduced limit of 17. Subsequent research has supported this refinement. (Abdelazim et al., 2020) conducted an extensive experimental testing of GFRP-RC columns and accordingly proposed a limit of 18. The same value was reported by another study (Syed & Goldack, 2024), which performed a numerical study on FRP-RC columns using an axial force ratio. Bujotzek & Waldmann (2025) further advanced the analytical framework by developing a buckling safety verification method that incorporates second-order effects as well as

long-term degradation from FRP creep. Their curvature-based approach offers a more refined and time-aware tool for establishing design slenderness limits than the current code criteria. Another study (Jaitrong et al., 2025) contributed experimental validation by testing square GFRP-RC columns over a range of slenderness ratios and reinforcement layouts, confirming that confinement type and stiffness significantly affect axial performance. However, it is important to note that all of these studies focus exclusively on column behaviour at ambient temperatures.

Fire resistance presents an additional challenge for GFRP-reinforced concrete columns. While concrete is non-combustible, the material and the internal reinforcement bars are thermally sensitive to extreme heat (Kodur & Dwaikat, 2010). GFRP degradation under elevated temperatures occurs progressively as the polymer matrix softens, transitions, and eventually decomposes. Multiple studies have identified the resin matrix as the thermal weak link in GFRP bars, with failure often occurring well before fibre degradation (Bisby, Kodur, et al., 2005; Chowdhury et al., 2009; Wang et al., 2007). The degradation process typically begins near the softening temperature (T_s), where the resin stiffness and mechanical properties begin to decline. As temperature increases, the material approaches the glass transition temperature (T_g), commonly reported between 110 °C and 180 °C for vinyl ester-based GFRP bars (S. Alsayed et al., 2012; Fleischhaker et al., 2014; Kodur & Bisby, 2005). At this point, the matrix undergoes a physical transformation from a glassy to a rubbery state, resulting in a rapid loss of the bond and stiffness. The critical temperature (T_{cr}), defined by (Wang & Kodur, 2005) as the point where approximately 50% of the FRP's tensile strength is lost and can no longer support the applied load, is often mentioned around 300–325 °C for GFRP (Al-Salloum, 2008; Thongchom et al., 2023). Beyond this, mechanical properties degrade at a slower rate until the resin decomposition temperature (T_d), typically between 350–450 °C (Chowdhury et al., 2009; Robert & Benmokrane, 2010). At this

point, the resin is largely eliminated, fibre bonding is lost, and the bar can no longer provide confinement or axial support. (Bazli & Abolfazli, 2020) further categorized this degradation into distinct thermal stages, documenting resin cracking, softening, fibre exposure, and eventual oxidation. Similar thermal-stage deterioration trends were experimentally confirmed, underscoring the dominant role of matrix breakdown in performance loss (Abushanab et al., 2024; S. Alsayed et al., 2012). These insights emphasize the importance of temperature-specific material characterization in assessing structural performance during and after fire exposure (Kodur & Harmathy, 2016).

Most prior research on fire-exposed FRP-RC members has focused on flexural elements such as beams and slabs, or small-scale specimens, offering limited insight into the behaviour of full-scale compression members under realistic fire conditions (Bai & Keller, 2011; Yu & Kodur, 2014). Numerical investigations have shown that elevated temperature exposure can lead to premature loss of stiffness and strength in FRP-RC members, particularly when cover is insufficient or reinforcement is poorly confined (Gao et al., 2013; Hajiloo & Green, 2019; Kodur & Baingo, 1998). However, such models often rely on simplified bond-slip relationships and thermal degradation assumptions that may not capture critical failure progression. Experimental findings have further highlighted the sensitivity of FRP-RC member performance, raising concerns about the adequacy of current design assumptions under fire. (Yu & Kodur, 2014) observed early bond failure in near-surface-mounted bars, and other post-fire evaluations (Bošnjak et al., 2024; Nigro et al., 2012) have shown that insufficient concrete cover and limited transverse reinforcement can severely compromise residual capacity.

While these studies provide valuable insight into flexural behaviour, they leave open questions about how such degradation mechanisms affect the performance of compression

members. Therefore, one study (Yoo & Choo, 2022) completed an experimental investigation on carbon FRP (CFRP)-RC columns, observing modest strength gains at 150 °C, followed by sharp degradation beyond 300 °C. Brittle failure had occurred, which was governed by bond loss and cover spalling, and residual capacity reduced to 55% at 450 °C. These trends are not fully captured by existing design provisions for axially loaded members. A separate study (Abd Elsalam et al., 2024) examined GFRP-RC columns under realistic fire exposure, specifically looking at fire resistance duration (time to failure) under varying concrete cover and applied load conditions. They reported that the resin was decomposed and subsequently eliminated the contribution of GFRP bars within the first hour. Together, these studies demonstrate that both CFRP and GFRP-reinforced columns are highly vulnerable under fire, with performance strongly influenced by early resin decomposition, detailing sensitivity, and confinement loss, which are not fully addressed in current design guidance.

Collectively, these unresolved challenges, including the contribution of GFRP to compressive axial capacity, the effect of slenderness, fire-induced degradation mechanisms, and the impact of sustained loading during fire exposure, underscore the need for large-scale experimental studies on GFRP-reinforced concrete columns under standardized fire conditions. Thus, to fill the gaps/challenges, seven large-scale square FRP-RC columns were constructed and tested. Testing parameters included variations in column length, fire exposure duration, and the presence of applied load during fire.

This chapter is organized into four sections. Section 1 (this section) introduces the need for improved understanding of the fire performance of GFRP-reinforced concrete columns. Section 2 outlines the experimental methodology, including specimen design, instrumentation, and testing

procedures. Section 3 presents the results and discussion of the findings, and Section 4 concludes the chapter, offering insights and future directions for research and design development.

2.2 Methodology

2.2.1 Column Design and Testing Parameters

This chapter investigates the fire performance of seven square GFRP-reinforced concrete columns, each with a cross-section of 200 mm $\times$ 200 mm and a height of either 1000 mm (short) or 1500 mm (slender), as seen in Figure 2.1. All columns were cast using a conventional concrete mix with a target compressive strength of approximately 35 MPa. The maximum aggregate size of 20 mm and a 30 mm concrete cover were used. The longitudinal reinforcement consisted of four No. 15 GFRP bars, while transverse reinforcement was provided by No. 10 mm GFRP ties, with six ties in the short columns and nine in the slender ones. This corresponded to tie spacings of 180 mm and 175 mm in the short and slender columns, respectively, which were considered essentially equivalent. The GFRP bars were manufactured using vinyl ester resin and had a minimum tensile strength of 1184 MPa, tensile modulus of elasticity of about 62.7 GPa, and ultimate strain of 1.89%. Column sizes were based on the maximum size allowed within the testing equipment. Specimen dimensions were selected to align with available testing equipment, where the furnace chamber limited the maximum column height to 1500 mm and the universal testing machine restricted practical cross-section size.

Testing parameters included variations in column length, fire exposure duration, and the presence of applied load during fire, as summarized in Table 2.1. Columns were exposed to either ambient conditions, one hour of fire, or two hours of fire. One slender column was subjected to an axial load of 200 kN during fire exposure to assess load-bearing performance under thermal stress. This value was selected to represent approximately 15–20% of the column's axial capacity, in line

with typical service-level loading assumptions used in fire testing (Kodur et al., 2013; Nair & Salem, 2020). To quantitatively capture the role of GFRP bars in axial capacity, one slender column was also cast without internal reinforcement. The designation of the columns consists of two parts. The first part describes the slenderness of the column (SH for short column, SL for long column). Whereas the second part describes the fire exposure period (F0 for ambient temperature, F1 for 1-hr fire exposure, F2 for 2-hr fire exposure). It is worth mentioning that the specimens with sustained load and cast without reinforcement (L) and (NR), respectively. For example, SL-F0-NR denotes a slender column tested under ambient temperature and had no longitudinal or transverse reinforcements.

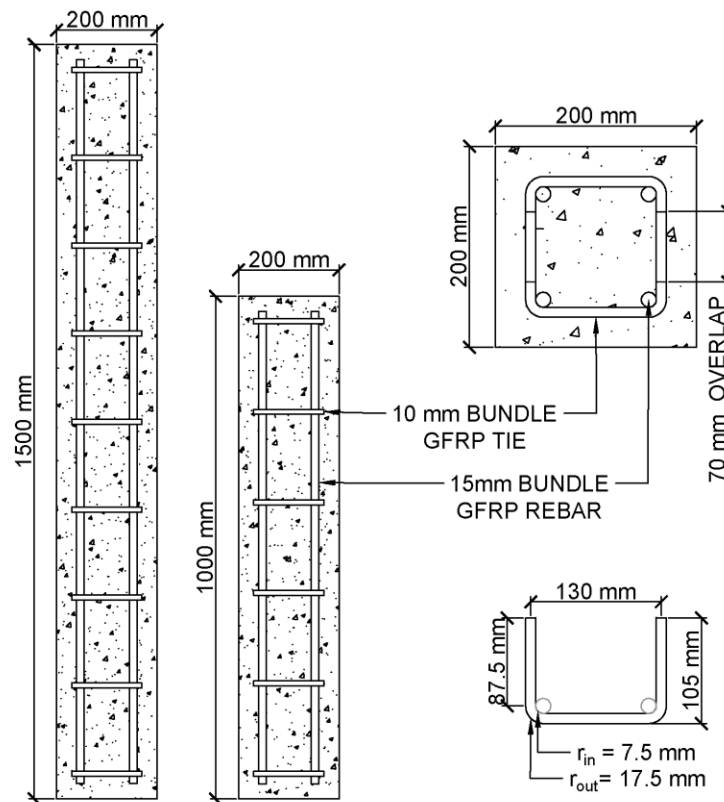


Figure 2.1 Reinforcement layout and specimen geometry for short (1000 mm) and slender (1500 mm) columns

Table 2.1 Specimen Testing Parameter Summary

Specimen ID	Length	Fire
SH-F0	1000 mm	No fire
SH-F1	1000 mm	1 hour
SL-F0	1500 mm	No fire
SL-F1	1500 mm	1 hour
SL-F2	1500 mm	2 hours
SL-F1L ^a	1500 mm	1 hour
SL-F0-NR ^b	1500 mm	No fire

^a The suffix L indicates that a 200 kN load was applied during fire testing

^b NR denotes that the specimen contains no reinforcement.

2.2.2 Specimen Fabrication

All GFRP reinforcement cages were preassembled using zip ties. First, the transverse ties, manufactured in two halves, were joined inside a custom-built 170 mm × 170 mm wooden jig to ensure consistent spacing and maintain the required 30 mm concrete cover. These ties were then secured to four longitudinal GFRP bars, forming square reinforcement cages. Concrete chairs were affixed to the bottom and sides of each cage to maintain appropriate cover during casting.

Formwork was constructed from plywood and dimensional lumber, designed to be modular for ease of assembly and disassembly. Once the cages were installed and sensors positioned (strain gauges), wires were threaded through pre-drilled holes in the back face of the formwork. The assembled forms were aligned on a flat, level casting surface and laterally braced using 2×4 lumber. Columns were cast vertically. Approximately three days after casting, the columns were removed from their formwork and wrapped with wet burlap, which was left on until the columns had reached 35 MPa.

Fire-tested specimens were instrumented with high-temperature K-type thermocouples (TCs), insulated with glass-fibre sheathing. Each thermocouple was attached to an auxiliary GFRP bar using zip ties to maintain correct alignment and positioning within the cross-section. Thermocouples were installed at 200 mm from each end of the column, with five sensors arranged diagonally across the cross-section (from top-left to bottom-right), as shown in Figure 2.2. The thermocouples were labelled sequentially from TC1 to TC10, where TC1 through TC5 were near the top and TC6 through TC10 were near the bottom. Short columns included all ten thermocouples, while slender columns were limited to TC6 through TC8 at the bottom due to wire length constraints. All sensor wires exited the column approximately 50 mm from the top surface. High-temperature thermocouple wires were rated up to 1000 °C and were cut to lengths extending approximately 1.2 meters beyond the furnace wall to allow connection to lower-temperature extension wires outside the fire chamber, then to the data acquisition system.

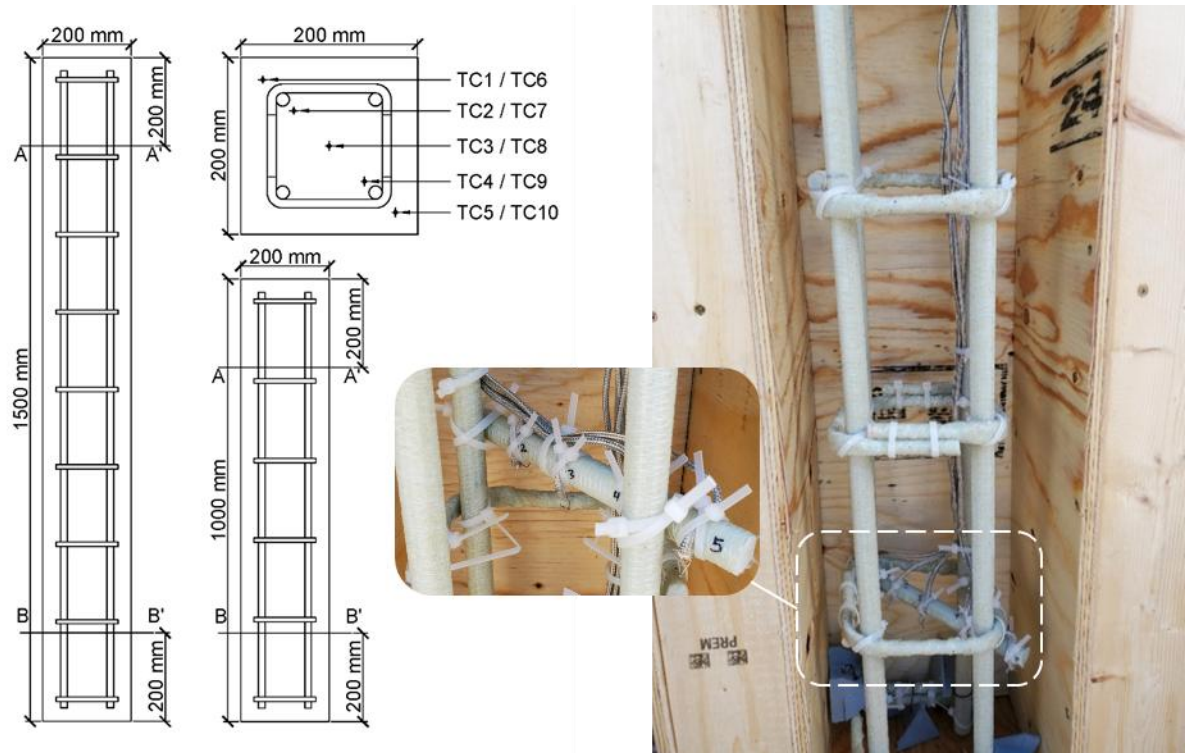


Figure 2.2 Thermocouple placement for both slender (1500 mm) and short (1000 mm) columns.

2.2.3 Fire Testing

Prior to fire exposure, all columns had been cured for approximately 6 months, to minimize shrinkage-related stresses and water content inside all specimens in order to reduce the risk of premature spalling during heating. To facilitate safe handling, each column was fitted with an anchor system. Fire testing was conducted at the Lakehead University Fire Testing and Research Laboratory (LUFTRL) in accordance with the CAN/ULC-S101 fire endurance standard (Canadian Standards Association and Underwriters' Laboratories of Canada, 2014), which defines a standardized time–temperature curve identical to ISO 834 and ASTM E119. The furnace temperature, T , at time t (in minutes) is given by Equation (2.1)

$$T = T_0 + 345 \times \log_{10}(8t + 1) \quad (2.1)$$

where T_0 is the initial furnace temperature, taken as 10 °C. This curve provides a consistent benchmark for evaluating fire resistance by simulating a controlled compartment fire. Target temperatures at 60 and 120 minutes are approximately 945 °C and 1049 °C, respectively.

A total of four furnace tests were performed: three involved one-hour exposures, while one test subjected a column to a two-hour fire. The fire exposure configurations were as follows: (1) two short columns (1000 mm) exposed for one hour; (2) three slender columns (1500 mm) exposed for one hour; (3) three slender columns exposed for two hours; and (4) one additional slender column exposed for one hour while subjected to a 200 kN axial load, corresponding to approximately 15% of its estimated ultimate axial capacity. After fire exposure, columns were allowed to air-cool within the furnace chamber and were left undisturbed for at least 24 hours before removal.

A view of the furnace setup is shown in Figure 2.3, illustrating the configuration used for Fire Test 3. Thermal insulation blankets were applied to the column ends to reduce heat loss

through furnace contact surfaces and to protect the high-temperature thermocouple wires and connection points exiting from the column tops. Although multiple columns were exposed during each furnace test, this study focuses exclusively on the compression behaviour of a single column from each group. The remaining specimens were reserved for a companion study evaluating different parameters, including the effectiveness of retrofitting fire-damaged GFRP-reinforced concrete columns. However, thermocouple data collected from all tested columns were analyzed to assess temperature distribution, evaluate test repeatability, and validate exposure conditions across specimens.



Figure 2.3 Furnace setup prior to Fire Test 3, showing a typical three-column configuration and thermal insulation around the column ends.

2.2.4 Compression Test Setup

Axial compression tests were carried out at the Lakehead University Structures Laboratory using a 500-kip Universal Testing Machine (UTM), as shown in Figure 2.4. All specimens were tested with pinned end conditions. To ensure uniform load distribution, each column was capped

with a steel plate and quick-set grout. Column ends were carefully measured and marked on the UTM plate, and the specimens were levelled to ensure concentric loading. A displacement-controlled loading rate of 0.5 mm/min was applied for all tests, with the exception of the ambient slender column (SL-F0), which was the first specimen to be tested and initially loaded at 1.5 mm/min before the rate was reduced to 0.5 mm/min. The results for SL-F0 were normalized to the standard loading rate to ensure comparability with the remaining specimens.



Figure 2.4 Typical axial compression test setup for a slender column in the 500-kip UTM.

2.2.5 Theoretical Compression Calculations

To benchmark the experimental results against code-based predictions, theoretical axial compression capacities were computed in accordance with CSA S806-12 (R2021), which references CSA A23.3-19 for column design provisions. Therefore, the calculations presented herein follow the requirements of CSA A23.3 as mandated by CSA S806. Since the standard does

not permit fibre-reinforced polymer (FRP) bars to contribute to axial compression resistance, the calculations excluded the reinforcement and assumed the capacity was derived entirely from the concrete. For each specimen, the nominal axial load capacity, denoted as P_{ro} , was calculated using Equation (2.2).

$$P_{ro} = \alpha_1 f'_c (A_g - A_{GFRP}) \quad (2.2)$$

In this equation, α_1 is the concrete stress block factor, f'_c is the specified compressive strength of the concrete, A_g is the gross cross-sectional area of the column, and A_{GFRP} is the total area of the GFRP reinforcement.

For slender columns, where instability may reduce load-carrying capacity, the nominal capacity, P_r , was further reduced using a second-order moment amplification approach based on Clause 10.15 of CSA A23.3 as per Equation (2.3)

$$P_r = \frac{P_{ro}}{1 + P_{ro}/P_{cr}} \quad (2.3)$$

Here, P_r is the slenderness-adjusted axial resistance, and P_{cr} is the Euler-based buckling capacity. Calculations assumed pin-ended conditions (i.e. $K = 1.0$), and the effective length, L_e was taken as the full column height. This adjustment reflects the increased vulnerability of taller columns to instability effects and enables more realistic comparisons to experimental data.

2.3 Results and Discussion

2.3.1 Load-Normalized Axial Displacement Relationship

Figure 2.5 presents the applied compression force versus column axial displacement for all specimens. Axial force was recorded directly from the UTM, while the axial displacement was obtained from the machine-reported vertical shortening and normalized by the total column height. Column stress is taken as the applied compression force divided by the cross-sectional area. The

structural behaviour of the tested columns varied significantly depending on the tested parameter, which includes fire exposure duration, column slenderness, applied load during exposure, and the presence of internal reinforcement. Since the load/stress-axial displacement relationship is nearly linear up to the peak load, the stiffness values were calculated as the slope between the measured load at 0 mm/m displacement and the peak load, expressed in kNm/mm.

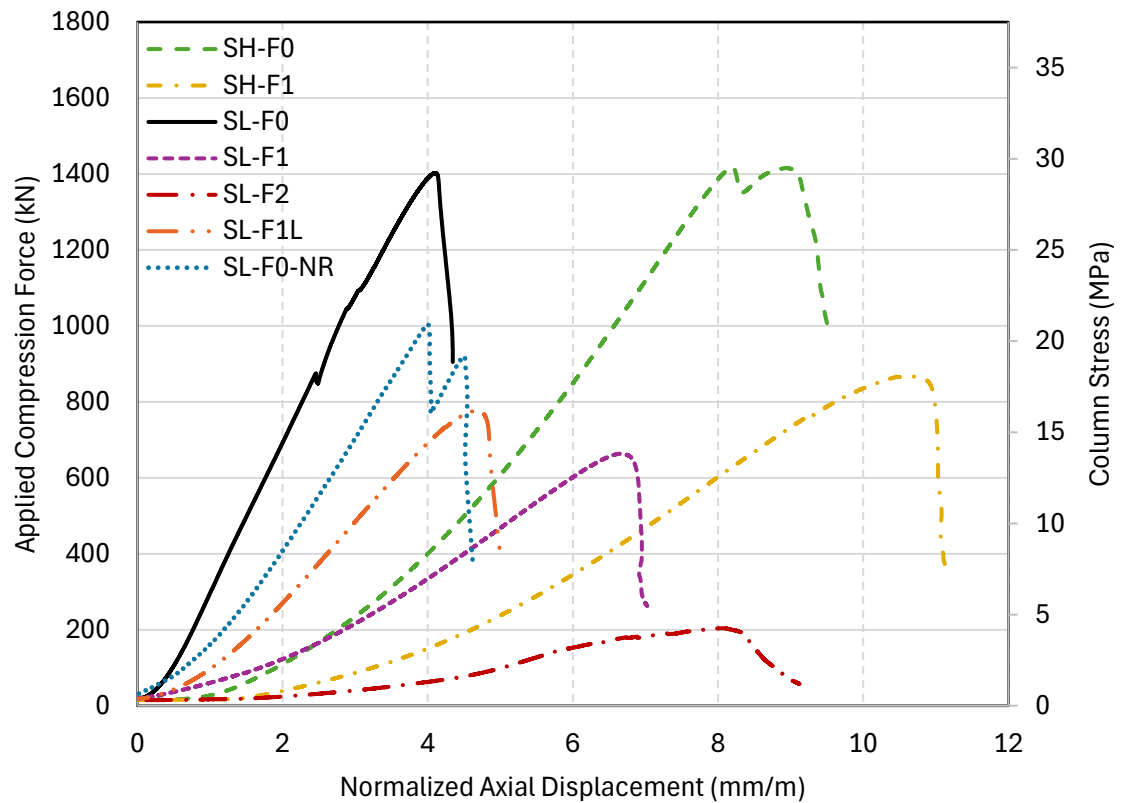


Figure 2.5 Applied compression force versus normalized axial displacement for all tested specimens.

The two ambient specimens, SH-F0 and SL-F0, achieved the highest peak loads of all tested columns, reaching 1418.7 kN and 1206.1 kN, respectively. SL-F0 exhibited the steepest initial slope, with a stiffness of 289.6 kN/mm/m, while SH-F0 showed a slightly more gradual initial response (171.1 kN/mm/m) but ultimately reached the highest overall capacity. After peak load, SH-F0 sustained a high level of axial force over an extended displacement range, indicating a relatively gradual loss of strength. In contrast, SL-F0 experienced a sharp and immediate drop in

load following peak, suggesting a more abrupt and brittle failure progression despite reaching a comparable maximum strength. These differences highlight the influence of column geometry on both stiffness and failure mode under ambient conditions.

The fire-exposed specimens showed progressive reductions in both strength and stiffness with increasing fire duration and slenderness. For example, the specimen SH-F1, the short column exposed to one hour of fire, reached a peak load of 866.7 kN and exhibited the highest displacement at failure (10.72 mm/m) among all specimens. Despite this high axial displacement capacity, the post-peak response was abrupt, with a steep drop in load and a relatively low stiffness of 79.5 kN/mm/m. On the other hand, the specimen SL-F1, a slender column subjected to the same fire duration, reached a peak load of only 664.0 kN and failed at a displacement of 6.67 mm/m. Although this displacement was still relatively high, it was followed by a sharper post-peak decline, indicating a more brittle failure with limited deformation after peak and lower energy absorption. Its initial stiffness was 96.8 kN/mm/m, slightly higher than that of SH-F1, indicating that both specimens experienced similar levels of stiffness degradation, with some influence from geometric differences. SL-F2, which was exposed to two hours of fire, experienced the most severe reduction in strength, reaching only 204.1 kN with a displacement of 8.05 mm/m. It exhibited the lowest stiffness of all specimens (23.5 kN/mm/m) and a flat initial slope, indicating extensive thermal degradation due to the formation of internal cracks. However, unlike SH-F1 and SL-F1, its post-peak decline was more gradual, suggesting a less localized and more progressive failure process.

The remaining two specimens, SL-F1L and SL-F0-NR, provide additional insight into the influence of fire test conditions and reinforcement configuration, respectively. SL-F1L, which was exposed to one hour of fire while under a sustained axial load of 200 kN, reached a peak load of

777.1 kN with a failure displacement of 4.67 mm/m. It recorded a stiffness of 162.5 kN/mm/m and exhibited a steeper post-peak drop and lower displacement capacity compared to its counterpart fire-exposed columns without sustained load. These results suggest that load application during fire exposure may accelerate internal damage and limit ductility, even though peak strength was only moderately affected. On the other hand, column SL-F0-NR, the unreinforced slender specimen tested under ambient conditions, achieved a relatively high peak load of 1003.9 kN but failed at the lowest displacement of all specimens (4.0 mm/m). It exhibited a stiffness of 243.3 kN/mm/m, second only to SL-F0, and showed a steep post-peak decline with minimal recovery. The absence of internal reinforcement likely contributed to its sudden, shear-like failure and limited residual strength, underscoring the role of GFRP bars in improving confinement capacity, which accordingly enhanced the full compressive capacity of the column and the post-peak behaviour.

The following subsections explore key parameters influencing column performance. Section 3.2 examines the thermal response of the columns during fire exposure. Section 3.3 evaluates the effects of fire duration and sustained loading during heating. Section 3.4 investigates how slenderness influences strength and failure mode. Section 3.5 focuses on the role of internal GFRP reinforcement, and Section 3.6 compares experimental results to theoretical predictions based on Canadian code provisions.

2.3.2 Thermal Response of Columns During Fire Exposure

The internal thermal response of each column was captured by thermocouples embedded at various depths. The thermal response profiles in Figure 2.6 demonstrate the progressive nature of heat penetration through the columns. Thermocouples logged values at one-second intervals, capturing both the heating and early cooling phases. Each curve includes 10 minutes of cooling

for the one-hour tests and 20 minutes for the two-hour tests. The furnace followed the CAN/ULC-S101 fire curve closely, reaching peak values of approximately 918 °C for the one-hour tests and 1004 °C for the two-hour test. These values are slightly below the theoretical targets of 945 °C and 1049 °C (based on an initial furnace temperature of 10 °C) but still within 96% of the standard curve, providing consistent exposure conditions across all tests. To generate the plotted curves, measurements from each thermocouple were averaged across all columns within a given fire test group. For example, TC1 readings were averaged from the two short columns in Fire Test 1 and the three specimens in Fire Tests 2 and 3. Thermocouples that recorded clearly erroneous data (e.g., peak temperatures below 50 °C) were excluded from the analysis.

During the initial heating phase, cover thermocouples (thermocouples near the cover) exhibited the steepest temperature rise, with early rates ranging from approximately 6 to 8 °C/min based on the time required to reach 200 °C. This rapid increase highlights the vulnerability of outer concrete regions to early thermal degradation. In contrast, thermocouples located deeper within the cross-section exhibited delayed and more gradual heating, taking approximately 30-50 minutes longer than the outer sensors to reach 200 °C in Fire Tests 1 and 2, and about 50 minutes longer in Fire Test 3. These delays resulted in pronounced thermal gradients between the inner and outer concrete temperatures, reaching peak differences of 432 °C, 386 °C, and 521 °C for Fire Tests 1, 2, and 3, respectively. In the one-hour tests, these peak differences occurred near the end of the heating phase (around 60 minutes), while in Fire Test 3, the maximum gradient appeared closer to the 90-minute mark. By the end of the two-hour exposure in Fire Test 3, however, nearly all sensors had exceeded 400 °C, indicating full thermal penetration and near-uniform internal heating.

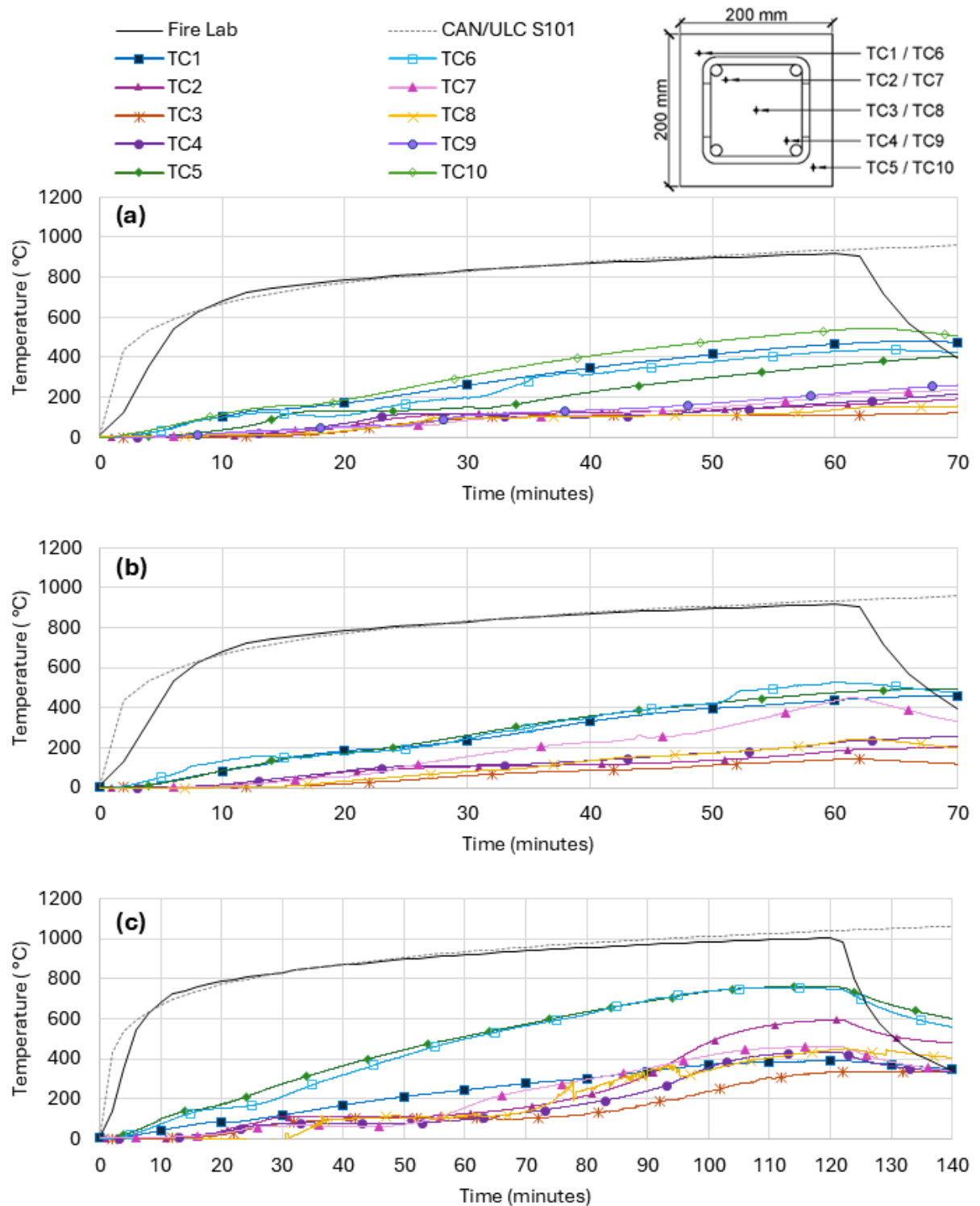


Figure 2.6 Measured thermocouple temperatures for (a) Fire Test 1: short columns exposed for one hour, (b) Fire Test 2: slender columns exposed for one hour, and (c) Fire Test 3: slender columns exposed for two hours. Note that Fire Tests 2 and 3 include only thermocouples TC1–TC8.

To simplify presentation and highlight cross-sectional thermal trends, thermocouples were grouped by depth within the column cross-section: those located within the concrete cover (TC1, TC5, TC6, and TC10) are referred to as cover thermocouples; those embedded just inside the reinforcement cage (TC2, TC4, TC7, and TC9) are referred to as inner cage thermocouples; and those positioned at the centre of the cross-section (TC3 and TC8) are referred to as centre thermocouples.

Figure 2.7 presents the average peak temperatures for each group by fire test. While some divergence was observed between sensors from different groups (cover, inner cage, centre), these discrepancies may reflect local furnace effects, such as flame non-uniformity, furnace wall turbulence, or small-scale spalling at the ends of certain specimens. Cover thermocouples recorded the highest average temperatures: 468 °C, 494 °C, and 664 °C for Fire Tests 1, 2, and 3, respectively. Inner cage thermocouples reached intermediate values (262 °C, 346 °C, and 477 °C), while centre thermocouples remained the coolest (188 °C, 195 °C, and 437 °C).

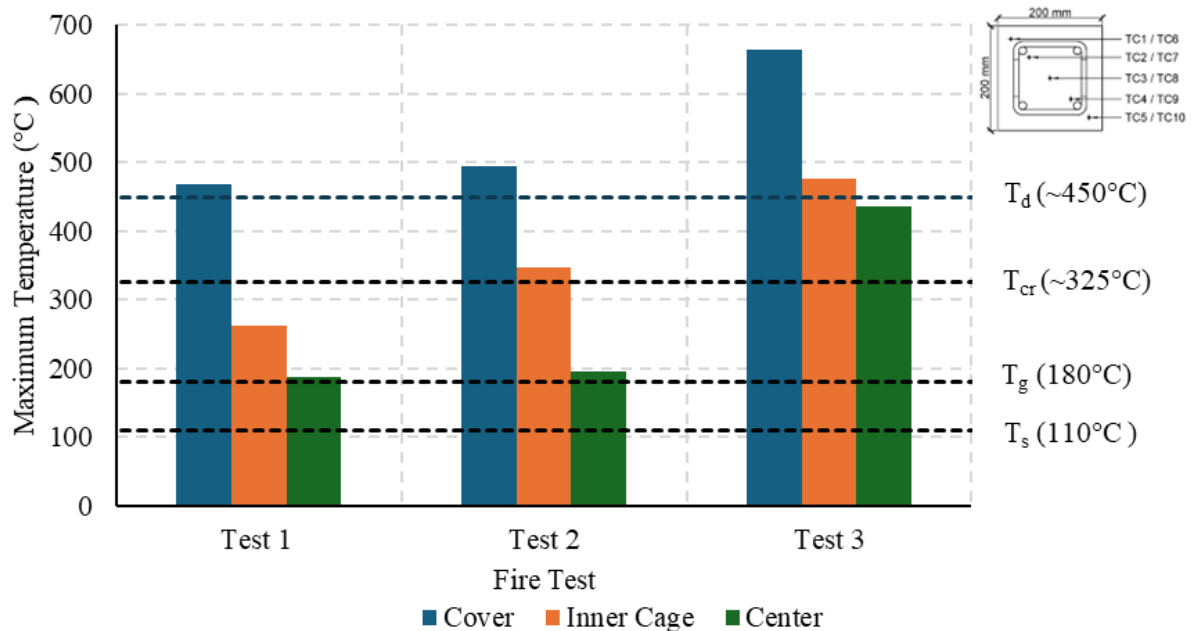


Figure 2.7 Peak Thermocouple Temperatures by Location and Fire Test

These temperatures can be evaluated against key thermal thresholds for GFRP degradation, as reported previously (S. Alsayed et al., 2012; Bisby, Kodur, et al., 2005; Fleischhaker et al., 2014; Wang & Kodur, 2005). The softening temperature (T_s , $\sim 110^\circ\text{C}$) marks the onset of polymer matrix softening. The glass transition temperature (T_g , $\sim 180^\circ\text{C}$ for vinyl ester-based GFRP) signals further loss of stiffness and bond. Both thresholds were exceeded at all locations in all tests. In all tests, cover thermocouples reached the softening temperature within the first 10-15 minutes, with inner cage sensors following shortly after. The glass transition threshold was generally exceeded at the inner cage level within 20-30 minutes in one-hour tests and much earlier in the two-hour exposure. Both thresholds were ultimately exceeded at all locations in all tests. The critical temperature (T_{cr} , $\sim 300\text{-}325^\circ\text{C}$) was surpassed at rebar depth and cover level in all tests except the center of the one-hour specimens. The resin decomposition temperature (T_d), ranging from $350\text{-}450^\circ\text{C}$, was exceeded at the cover level in all tests and at the rebar level and center during the two-hour exposure, suggesting widespread material breakdown in Fire Test 3.

Together, these thermal results confirm that while the 30 mm concrete cover provided meaningful protection during one-hour exposures, the extended heating in Fire Test 3 resulted in near-complete thermal penetration, including core-level degradation. It should be noted that the column dimensions were partially governed by the available testing equipment, which limited specimen size but still allowed for representative evaluation of cover performance and internal temperature gradients. This progression in thermal severity aligns with the reductions in axial capacity observed in fire-exposed specimens and provides a critical backdrop for evaluating the influence of fire duration, sustained loading, and slenderness discussed in the following sections.

2.3.3 Impact of Fire Duration and Loading During Fire

The axial capacities of the fire-exposed columns reveal the significant influence of fire duration on structural performance. Among the short specimens, the fire-exposed column (SH-F1) retained 61% of the axial capacity of its unexposed counterpart (SH-F0), indicating a moderate loss in strength after one hour of fire. In contrast, the slender columns experienced more severe degradation: the one-hour fire-exposed specimen (SL-F1) retained only 55% of the capacity of the ambient slender column (SL-F0), while the two-hour exposed column (SL-F2) retained just 17%. These results reflect the expected progression of material degradation with increasing fire duration, consistent with rising internal temperatures and prolonged thermal exposure.

Interestingly, the slender column subjected to fire under a sustained axial load of 200 kN (SL-F1L) reached 65% of SL-F0's capacity, which is 10% higher than the unloaded counterpart (SL-F1). Although this value was originally selected to represent 15-20% of the column's theoretical capacity, the higher-than-expected concrete strength (50 MPa) meant it ultimately corresponded to only 13% of the unadjusted theoretical capacity (P_{ro}) and 14% of the slenderness-adjusted value (P_r). Due to equipment limits, the 200 kN load could not be increased to reflect the higher concrete strength, resulting in a lower-than-expected load ratio. In addition to the previous reason, the sustained load during the exposure might contribute to reducing the progression of thermal cracks that are perpendicular to the load direction. Nevertheless, including a loaded specimen provided valuable context for interpreting the overall fire response, and results suggest that even modest axial loading during heating may influence damage development and reduce deformation capacity.

2.3.4 *Impact of Slenderness*

The role of column slenderness on axial performance was assessed by comparing short (1000 mm) and slender (1500 mm) specimens tested under both ambient and fire-exposed conditions. In the absence of thermal damage, slenderness had a measurable influence on capacity: the slender ambient column (SL-F0) reached 1202.5 kN, approximately 85% of the capacity of the short column (SH-F0), which failed at 1418.7 kN. This highlights the impact of second-order effects when columns are loaded and fully confined, and when material properties remain intact.

Under fire exposure, however, the influence of slenderness became more pronounced. After one hour of heating, the short column (SH-F1) retained 61% of its ambient strength, while the slender column (SL-F1) retained only 55%. This reduction translates to a fire-exposed slender column achieving just 77% of the capacity of its short counterpart. The greater performance loss observed in slender specimens is attributed to the compounded effects of thermal degradation and stability loss. Thermal data from Section 3.2 confirms that the slender columns in Fire Test 2 experienced higher internal temperatures than the short columns in Fire Test 1, despite identical furnace exposure.

The most pronounced difference occurred at the reinforcement level, where peak temperatures at the inner cage depth increased by over 80 °C between Fire Tests 1 and 2, highlighting the greater susceptibility of slender members to accelerated thermal penetration at rebar depth. The elevated thermal exposure is likely a result of the higher surface-to-volume ratio in slender columns, which facilitates faster heat ingress and deeper internal heating. Taken together, these findings indicate that slenderness becomes significant under ambient temperature, but it is a more critical factor when thermal degradation is introduced, particularly for members relying on confinement and stability to maintain load-carrying capacity.

2.3.5 *Impact of Rebar*

To assess the role of GFRP reinforcement in axial compression, the performance of reinforced and unreinforced slender columns was compared under ambient conditions. The unreinforced specimen (SL-F0-NR) reached a peak load of 1003.9 kN, approximately 83% of the capacity of its reinforced counterpart. This difference suggests that reinforcement had some influence on column behaviour. It raises the question of whether the additional capacity stemmed from the longitudinal bars directly contributing to axial resistance or from improved confinement provided by the transverse reinforcement.

To better interpret this distinction, the post-test condition of GFRP bars under different thermal exposures was examined, as shown in Figure 2.8. The control bar (not exposed to fire and not tested) remained smooth and intact, showing no visible deterioration. The bar retrieved from the ambient-condition specimen exhibited minor surface roughening but remained structurally sound. After one hour of fire exposure, the vinyl ester resin had visibly degraded, and longitudinal splitting along the fibres was observed. Following two hours of exposure, the resin had fully decomposed, the fibres appeared blackened, and the bar could be broken apart with minimal effort. These results are consistent with the internal temperature data presented in Section 2.3.2, where thermocouples near the reinforcement depth exceeded critical thermal thresholds for matrix degradation, bond loss, and resin decomposition.



Figure 2.8 Condition of GFRP reinforcement before and after exposure. a) Control GFRP bar with no exposure and no test, b) GFRP bar extracted from ambient-temperature specimen, c) GFRP bar extracted from specimen exposed to 1-hour standard fire, and d) GFRP bar extracted from specimen exposed to 2-hour standard fire

It should be noted that only qualitative observations of the bar condition were made in this study. No post-fire mechanical testing of extracted bars (such as residual tensile strength or bond pull-out) was performed, as the experimental program already encompassed a broad scope. Residual tensile and bond behaviour of fire-exposed GFRP bars has, however, been investigated in previous studies on flexural members, which consistently confirm progressive bond loss beginning near the glass transition temperature (T_g) and becoming severe once the critical (T_{cr}) and decomposition (T_d) thresholds are exceeded (e.g., Bošnjak et al., 2024). The present

observations, therefore, complement existing material-level findings by demonstrating how bar deterioration manifests within the behaviour of full-column specimens.

The clear deterioration of GFRP under elevated temperatures reinforces the need for cautious treatment of reinforcement contributions in fire scenarios. Design approaches that assume intact rebar performance post-fire may overestimate residual capacity. Theoretical calculations in the next section provide further insight.

2.3.6 Comparison to Theoretical Calculations

To contextualize the experimental results, axial capacities were compared against theoretical predictions based on CSA A23.3 (as said equations for unreinforced concrete, as summarized in Table 2.2. These predictions, outlined in Section 2.2.5, were computed using measured concrete strengths from companion cylinder tests conducted in close temporal proximity to column testing. Reinforcement was excluded from calculations, which were in line with Canadian design provisions for GFRP-reinforced columns, which assume no axial contribution from FRP bars.

Table 2.2 Experimental and Theoretical Capacities of GFRP-RC specimens

Specimen ID	Concrete Test-Day Strength	Column Experimental Capacity, P_{ex}	Column Theoretical (Ambient) Capacity		Experimental-Theoretical Capacity	
			P_{ro}	P_r	P_{ex}/P_{ro}	P_{ex}/P_r
	(MPa)	(kN)	(kN)	(kN)		
SH-F0	50	1418.7	1522.6	-	93%	-
SH-F1	46	866.7	1411.6	-	61%	-
SL-F0	39	1206.1	1212.9	1137.3	99%	106%
SL-F1	36	664.0	1126.0	1058.6	59%	63%
SL-F2	36	204.1	1126.0	1058.6	18%	19%
SL-F1L	50	777.1	1522.6	1415.1	51%	55%

SL-F0-NR	50	1003.9	1522.6	1415.1	66%	71%
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Under ambient conditions, experimental capacities were generally consistent with theoretical values. The short column (SH-F0) reached 93% of its predicted capacity, while the slender column (SL-F0) exceeded its prediction by 6%. The latter result suggests that the presence of GFRP reinforcement, though not captured in design equations, may assist the concrete in reaching its full axial capacity, particularly in slender elements. In contrast, the short column's inability to reach its unreinforced prediction implies that, unless substantial confinement is present (as seen in other studies), excluding GFRP bars from axial resistance remains a conservative design approach.

Fire-exposed specimens exhibited substantial reductions relative to theoretical predictions. The short column, SH-F1, reached 61% of its theoretical capacity, aligning well with its 61% retention relative to SH-F0 (Section 3.3). For the slender columns, this agreement was somewhat more complex. SL-F2 showed strong alignment between comparison methods, retaining 17% of its ambient capacity and 19% of its theoretical capacity. However, for the one-hour fire-exposed specimens, SL-F1 and SL-F1L showed an inversion: SL-F1 retained 55% of the ambient capacity but reached 63% of its theoretical capacity, while SL-F1L retained 65% of the ambient capacity but only 55% of its theoretical capacity. This reversal is primarily attributed to the higher concrete strength in SL-F1L, which raised its theoretical benchmark. While some variation is expected due to differences in concrete strength, the overall consistency in degradation patterns across both comparison methods strengthens confidence in the observed performance trends.

Likewise, the ambient unreinforced column (SL-F0-NR) reached 71% of its theoretical prediction, compared to 83% of the experimental capacity of its reinforced counterpart (SL-F0). This difference highlights the added confinement and stiffness provided by internal GFRP

reinforcement, even though its compressive contribution is not explicitly considered in current design equations.

Still, the consistency across ambient and fire-exposed specimens reinforces the broader conclusion that GFRP reinforcement does not offer reliable post-fire axial capacity, and that residual column performance is primarily governed by concrete degradation and instability effects.

2.4 Conclusion

This chapter evaluated the thermal and structural performance of GFRP-reinforced concrete columns exposed to fire. Seven full-scale specimens were tested to examine the effects of fire duration, slenderness, sustained axial load during fire, and internal reinforcement on residual axial capacity. Key findings are summarized below:

- Fire exposure significantly reduced axial capacity: The short fire-exposed column (SH-F1) retained 61% of its ambient counterpart's (SH-F0) capacity, while the one-hour and two-hour fire-exposed slender columns (SL-F1 and SL-F2) retained 55% and 17% of the ambient slender column's (SL-F0) capacity, respectively. These reductions correspond to internal temperatures exceeding critical thresholds for matrix softening, bond loss, and resin decomposition, with thermocouples recording peak thermal gradients up to 521 °C between cover and core.
- Although slenderness had less effect under ambient conditions—with the slender column reaching 85% of the capacity of its short counterpart—it had a more pronounced impact after fire exposure, where the fire-exposed slender column failed at only 77% of the corresponding short specimen's capacity. Internal temperatures at reinforcement level were over 80 °C higher in slender columns, confirming accelerated heat penetration due to their greater surface-to-volume ratio.

- The loaded column (SL-F1L) retained 65% of SL-F0's capacity, 17% higher than the unloaded SL-F1, indicating that modest axial load during fire may influence damage development and displacement capacity. On the other hand, the unreinforced slender column (SL-F0-NR) reached 83% of the strength of its reinforced counterpart, suggesting a limited role of the bars in axial compression, likely through confinement or stiffness effects.
- Post-test inspection revealed matrix loss, fibre splitting, and loss of structural integrity after one-hour and two-hour exposures, consistent with internal temperature data. Ambient columns closely matched or exceeded unreinforced theoretical capacities, while fire-exposed columns fell well below their predicted strength, validating the exclusion of FRP contributions in compression design under fire.

These findings underscore the importance of accounting for fire duration, slenderness, thermal gradients, and reinforcement degradation in the design and post-fire assessment of GFRP-reinforced concrete columns. In addition, these findings would pave the way for the importance of applying retrofitting techniques for deficient/deteriorated columns (e.g., FRP wraps).

Chapter 3 - Behaviour of Fire-Damaged GFRP-Reinforced Concrete

Columns Retrofitted with CFRP Wraps

3.1 Introduction

Structural elements are commonly exposed to a range of environmental stressors throughout their service life, with fire representing one of the most critical hazards (Gernay, 2024; Malik et al., 2021). Although reinforced concrete (RC) exhibits favourable fire resistance due to its low thermal conductivity and non-combustible nature, it still undergoes degradation in strength and stiffness when subjected to elevated temperatures (Balaji et al., 2016; Qureshi et al., 2020). Columns, as primary load-bearing elements, are particularly vulnerable, as a loss in their axial capacity can lead to partial or complete structural collapse (Alhadid & Youssef, 2016; Dotreppe et al., 1997). Replacing fire-damaged columns is often impractical due to the associated cost, time constraints, and operational demands, particularly in critical infrastructure, causing rehabilitation methods to become essential (Raza et al., 2019). Among these rehabilitation methods, confinement techniques - often referred to as jacketing/wrapping - have proven effective in enhancing axial strength, ductility, and deformation capacity by limiting lateral expansion and delaying the onset of instability (Considère, 1903; Montuori et al., 2013; Sheikh & Uzumeri, 1982).

Various jacketing methods have been developed to restore the load-carrying capacity of fire-damaged or deficient concrete columns. Concrete jacketing, one of the earliest techniques, enhances axial and flexural strength by encasing the existing member in a new layer of reinforced concrete mechanically anchored to the original section (Campione et al., 2014; Tayeh et al., 2019). While durable and familiar, this method significantly increases cross-sectional area, mass, and stiffness, making it unsuitable in space-constrained applications. Alternatively, steel jacketing

offers high ductility and has been used effectively to increase both strength and deformation capacity (Fakharifar et al., 2016; Xiao, Y., Wu, 2000). However, steel jackets are susceptible to corrosion and still contribute considerable additional weight. In contrast, fibre-reinforced polymer (FRP) jacketing systems provide a lightweight, high-strength alternative with minimal increase in dimensions. FRP can be applied through wet lay-up or pre-cured systems, both of which allow for rapid installation and effective confinement. These advantages have positioned FRP wrapping, particularly using carbon FRP (CFRP), as a promising technique for structural rehabilitation, especially when rapid deployment, reduced added mass, or limited access are critical considerations (Bisby, Dent, et al., 2005; Farrokh Ghatte et al., 2019).

FRP jacketing systems are most commonly applied as externally bonded fabrics or sheets wrapped around the perimeter of concrete columns to enhance confinement capabilities and axial performance. Although other FRP-based retrofit methods, such as near-surface mounted reinforcement, exist (De Lorenzis & Teng, 2007), external wrapping has become the most widely adopted due to its simplicity and versatility (Bilotta et al., 2011; Koczynia et al., 2008; Pohoryles et al., 2019; Triantafillou, 1998). Early studies demonstrated that confinement concepts developed for steel jackets could be extended to FRP systems (Fardis & Khalili, 1981, 1982), supporting broader use in seismic strengthening and rehabilitation of aging infrastructure (Demers & Neale, 1999; Spoelstra & Monti, 1999). While external wrapping using FRP has proven effective in many structural applications, existing confinement models remain based on idealized assumptions. Current design guidance for FRP-confined concrete, such as CSA S806-12 (R2021) and ACI PRC-440.2-23, provides confinement models based on idealized assumptions that may not account for damage or deterioration, highlighting the need for further validation under post-fire conditions.

The performance of CFRP-wrapped concrete columns is influenced by a range of factors, including material properties, geometry, and loading conditions (H.-L. Wu et al., 2009). Among material parameters, the type of fibre plays a critical role. CFRP has consistently outperformed alternatives such as GFRP and polyester FRP (PFRP) in enhancing compressive strength, with one study reporting significantly higher gains for CFRP-confined columns (Kumar & Reddy, 2021). In addition to fibre type, effective confinement depends on strong bonding between the concrete, adhesive, and FRP layers. Factors such as fibre orientation, adhesive thickness, surface preparation, and environmental durability, including moisture exposure and UV degradation, can significantly affect long-term performance (Elbakry & Tarabia, 2016; Harries & Carey, 2003). Geometric characteristics, particularly cross-sectional shape, also influence retrofit effectiveness. While circular columns typically respond well to confinement, square and rectangular sections can exhibit strain localization and reduced efficiency unless corner radii are sufficiently rounded; large-scale testing has shown that increasing the radius improves both strength and strain uniformity (Shan et al., 2019; Y.-F. Wu & Wang, 2009). Finally, loading conditions are critical; while FRP jackets are most effective under concentric axial loads, their efficiency may decrease under eccentric loading, which can alter the confinement mechanism and failure mode (Parvin & Wang, 2001).

While much of the research on FRP confinement has focused on externally applied systems, FRP materials have also been increasingly adopted as internal reinforcement. Glass fibre-reinforced polymer (GFRP) bars are increasingly being adopted as internal reinforcement in concrete structures due to their high strength-to-weight ratio, corrosion resistance, and long-term durability in aggressive environments (Elmessalami et al., 2019). As the use of GFRP-reinforced concrete (GFRP-RC) expands, particularly in marine, de-icing, and chemically exposed

infrastructure, understanding its performance under extreme conditions has become a growing research priority (Robert & Benmokrane, 2010). One of the critical concerns is the vulnerability of GFRP to elevated temperatures. Unlike steel, GFRP begins to lose strength and bond capacity at relatively low temperatures due to resin softening and fibre degradation (S. Alsayed et al., 2012; Wang & Kodur, 2005). Exposure to fire can result in severe reductions in axial capacity and overall structural integrity, especially in compression-dominant elements such as columns (Abd Elsalam et al., 2024). Despite this known vulnerability, research on post-fire rehabilitation strategies for GFRP-RC columns remains limited.

Although CFRP retrofitting has been studied in several areas, such as flexural strengthening of FRP-RC beams and confinement of steel-RC columns (Jafarzadeh & Nematzadeh, 2022; Shayanfar et al., 2024), its application to GFRP-RC columns remains limited. This is especially true under post-fire conditions, where no experimental studies are known to have evaluated whether CFRP jacketing can restore axial capacity. This gap is critical because thermal degradation of GFRP reinforcement can result in substantial strength loss. Based on these aforementioned limitations, this chapter presents the second phase of an ongoing research program at Lakehead University. The first phase (McTavish et al., forthcoming) focused on quantifying the residual strength of fire-damaged columns without retrofitting, while this phase explores the potential for strength recovery using external CFRP confinement.

This chapter is organized into four sections: Section 1 outlines the background and research motivation; Section 2 presents the experimental methodology; Section 3 details the test results and analysis; and Section 4 concludes with key findings and directions for future research.

3.2 Methodology

3.2.1 *Testing Objective and Parameters*

The objective of this study is to investigate the axial performance of GFRP-reinforced concrete (GFRP-RC) columns retrofitted with externally bonded CFRP wrapping following exposure to elevated temperatures. To reflect real-world post-fire rehabilitation scenarios, the twelve columns tested under axial compression included a range of initial exposure conditions: four ambient-temperature columns, five exposed to 1-hour fire durations, and three exposed to 2-hour fire durations. These fire exposure levels were treated as initial damage states, rather than experimental variables.

The primary testing parameter was the thickness of CFRP wrapping, with three configurations evaluated: no wrapping, one layer of CFRP (1.3 mm), and two layers of CFRP (2.6 mm). A secondary parameter was column slenderness, with three short columns (1000 mm) and nine slender columns (1500 mm) included to provide a limited comparison. One short column was also retrofitted to assess whether CFRP confinement was similarly effective in non-slender elements. While a two-layer wrap was originally planned for one of the ambient specimens to match the fire-exposed configuration, this was reconsidered due to concerns about approaching the capacity limit of the testing machine (2224 kN). Therefore, two ambient specimens were retrofitted with a single CFRP layer (SL-F0-C1a and SL-F0-C1b). Specimen dimensions were also partially dictated by equipment capacity, as the furnace chamber height limited column length to 1500 mm and the universal testing machine restricted practical cross-section size.

Table 3.1 summarizes the specimen identifiers, column lengths, fire exposure durations, and CFRP wrapping configurations included in this study. The ultimate capacities of the five unwrapped specimens were previously reported as part of a companion investigation focused on

fire exposure effects (McTavish et al., forthcoming). No additional testing was performed on these columns; their axial capacities are presented here as reference benchmarks for evaluating the effectiveness of CFRP rehabilitation. The columns are named in three parts. The first part describes the slenderness of the column (SH for short column, SL for long column). Whereas the second part describes the fire exposure period (F0 for ambient temperature, F1 for 1-hr fire exposure, F2 for 2-hr fire exposure). The third part represents the number of CFRP layers (C0 for no layers, C1 for one layer, and C2 for two layers of CFRP). For example, SL-F1-C1 denotes a slender column tested after 1-hour fire exposure and retrofitted with one layer of CFRP wrap.

Table 3.1. Test matrix for CFRP retrofitting of fire-exposed GFRP-RC columns

Specimen ID	Length	Fire Duration	CFRP Wrapping
SH-F0-C0	1000 mm	No fire	None
SH-F1-C0	1000 mm	1 hour	None
SH-F1-C1	1000 mm	1 hour	1 layer
SL-F0-C0	1500 mm	No fire	None
SL-F0-C1a	1500 mm	No fire	1 layer
SL-F0-C1b*	1500 mm	No fire	1 layer
SL-F1-C0	1500 mm	1 hour	None
SL-F1-C1	1500 mm	1 hour	1 layer
SL-F1-C2	1500 mm	1 hour	2 layers
SL-F2-C0	1500 mm	2 hours	None
SL-F2-C1	1500 mm	2 hours	1 layer
SL-F2-C2	1500 mm	2 hours	2 layers

*This reference specimen did not apply two layers of CFRP due to concerns related to approaching the full capacity of the UTM machine

3.2.2 Specimen Design and Fire Exposure

Specimens were fabricated with nominal cross-sectional dimensions of 200 mm × 200 mm and lengths of either 1000 mm (short) or 1500 mm (slender). Each column was reinforced with four

bundled 15 mm diameter sand-coated GFRP bars placed near the corners for longitudinal reinforcement, and 10 mm diameter bundled GFRP ties. The clear concrete cover was 30 mm in all directions, as depicted in Figure 3.1. Reinforcement detailing was held uniform across all specimens to maintain consistency in confinement conditions and to isolate the effects of fire exposure and subsequent CFRP rehabilitation.

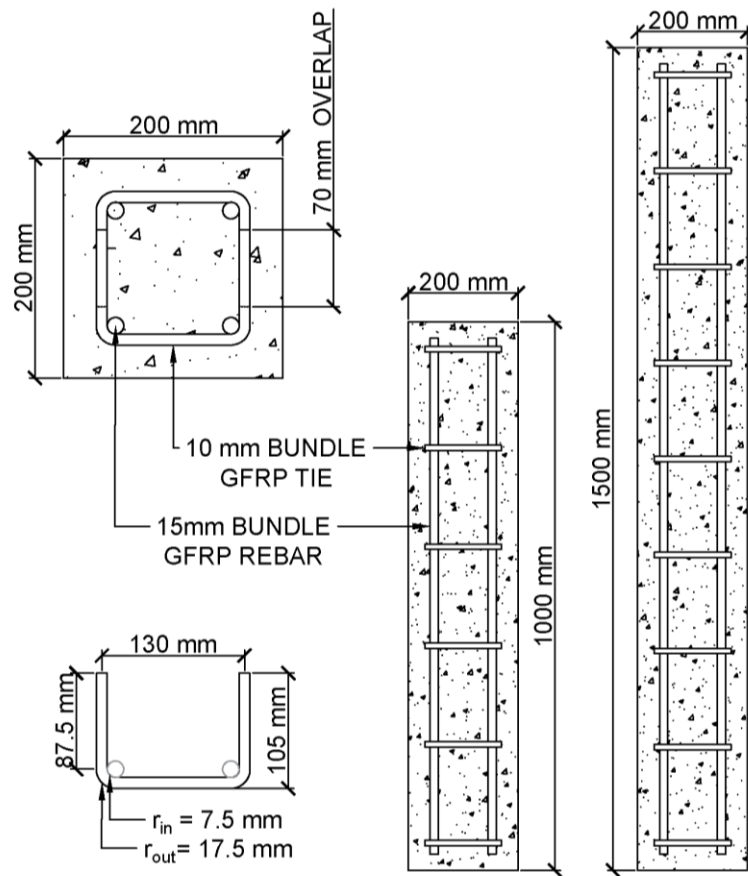


Figure 3.1 Column geometry and internal reinforcement configuration for GFRP-RC specimens

Manufacturer-reported mechanical properties included a minimum tensile strength of 1100 MPa and a tensile modulus of approximately 60 GPa. All columns were cast vertically in plywood formwork and consolidated using internal vibration. Concrete was supplied by a local ready-mix plant using a standard 35 MPa mix design with a maximum aggregate size of 20 mm. Formwork

was removed approximately three days after casting, and specimens were wrapped in wet burlap to maintain surface moisture. The burlap was removed once the concrete reached a compressive strength of 35 MPa, after which all specimens were stored under ambient laboratory conditions for approximately six months prior to testing.

Instrumentation was integrated during fabrication. For the ambient-temperature specimens, surface-mounted electrical resistance strain gauges were installed at mid-height on each of the four longitudinal GFRP bars to monitor axial strain during compression testing. For fire-exposed columns, high-temperature K-type thermocouples insulated with glass-fibre sheathing were installed at multiple depths within the cross-section to monitor internal temperature development during furnace exposure. These were tied to auxiliary GFRP bars and fixed into position within the cage.

Fire exposure was conducted at the Lakehead University Fire Testing and Research Laboratory (LUFTRL) using a vertical gas furnace configured to apply heat to the sides of each column. The furnace followed the CAN/ULC-S101 standard time-temperature curve. Columns were placed vertically in the furnace and remained unloaded during testing, but were restrained laterally to prevent tipping. Internal temperature development was monitored using embedded K-type thermocouples installed during fabrication. Additional details on thermocouple installation and thermal response are provided in the companion study (McTavish et al., forthcoming).

3.2.3 CFRP Retrofitting Procedure

3.2.3.1 CFRP Wrap and Adhesive Properties

Sikawrap-1400C (Sika Canada Inc., 2017b) is a high-strength, high-modulus, lightweight, unidirectional carbon fibre fabric which can be used for shear, confinement, and flexural strengthening. It came in a roll 0.6m wide.

To secure the CFRP Sikawrap fabric to the concrete column specimens, Sikadur-300 epoxy (Sika Canada Inc., 2017a) is applied using the wet application method. The epoxy comprises two components (100% solids) that are mixed at a 2.9:1 weight ratio at the time of application. Sikadur-300 epoxy is high temperature, high abrasion, and shock resistant and can resist high amounts of creep under permanent loads. While it has a longer pot life, higher temperatures or larger mixed quantities can lower the time in which it can be applied.

3.2.3.2 Column Preparation

To ensure a consistent and reliable bond between the CFRP wrap and the concrete surface, all retrofitted columns were prepared by rounding their corners to a radius of 20 mm. This is a common practice in confinement strengthening applications to avoid stress concentrations and improve confinement effectiveness, as shown in previous experimental work (e.g., Shan et al. 2019).

Corner rounding was achieved using a handheld concrete grinder, and the radius was verified using a custom wooden jig with a 20 mm cut-out. For fire-exposed columns, the surface condition was often brittle and uneven due to thermal degradation. The fine cement paste could be easily removed, but coarse aggregates remained, leaving jagged or irregular edges. These were repaired using a commercial concrete patching compound applied to pre-wetted surfaces. The patch was shaped using the same 20 mm jig to restore a smooth and consistent geometry. Repairs were completed in two stages: two corners were patched and left to cure overnight before the column was rotated to complete the remaining corners.

In addition to corner repair, any large-spalled regions or surface voids were patched to improve wrap adherence. Thermocouple wires embedded during fire testing were trimmed and ground flush with the surface to avoid interference with the CFRP application.

3.2.3.3 *CFRP and Epoxy Preparation*

SikaWrap-1400 C fabric was cut into 1.0 m lengths (600 mm width) prior to installation. For slender columns (1500 mm), three sheets per layer were required to provide full height coverage with overlap. Short columns (1000 mm) required two sheets. A longitudinal overlap of 150 mm was maintained between adjacent sheets, consistent with internal lab practice and ACI 440.2R-23, which recommends a minimum overlap of 6 inches (152 mm) for continuous wraps.

The required quantity of epoxy was determined based on the total surface area of CFRP to be applied and the corresponding fabric weight. Each 1.0 m $\times$ 0.61 m sheet was weighed (approximately 863 g per sheet), and an approximate resin-to-fibre weight ratio of 1:1 was used as a practical guide to ensure complete saturation. Additional epoxy was included for priming the concrete surface, using a surface coverage estimate of 3 m²/L. This approach allowed the resin volume to be tailored to the application without relying on fixed batch sizes.

3.2.3.4 *CFRP Wrapping Process*

The wrapping process involved a manual wet lay-up technique using pre-cut CFRP sheets and epoxy resin. The goal was to ensure uniform saturation of the fabric, complete contact with the concrete substrate, and effective confinement through proper installation and curing procedures, as depicted in Figure 3.2. The following steps were carried out:

1. CFRP sheets were laid flat on a polyethylene-covered table beside the columns to allow for manual saturation prior to placement.
2. Epoxy was mixed in small quantities. Components A and B were stirred individually, then combined at a 2.9:1 weight ratio and blended using a spiral-blade mixer attached to a low-speed drill (500 rpm) for approximately five minutes.

3. Epoxy was applied to all four sides of the column using paintbrushes. The column, including the underside, was rotated during application to ensure consistent coverage.
4. Each CFRP sheet was saturated on both sides using paintbrushes and rollers. Once saturated, the sheet was rolled up to facilitate handling and minimize contamination.
5. Approximately 30 minutes after priming, once the epoxy on the column surface became tacky, the saturated CFRP sheets were wrapped around the column.
6. Wrapping began at the center of one face and continued in a continuous path around the column. This method helped avoid bond failure or lifting at the corners. Each sheet was smoothed repeatedly using a modified plastic roller to ensure full contact and to remove any entrapped air.
7. For columns requiring two CFRP layers, the procedure was repeated without reapplying epoxy to the concrete surface. The second layer was applied while the first remained tacky to ensure proper interlayer bonding. The installation sequence was as follows: the center sheet of the first layer was applied first, followed by the two outer sheets. Once the first layer was fully in place, the center sheet of the second layer was applied, followed by its outer sheets. This order of installation helped minimize resin buildup and potential gaps in the overlap zones, particularly near the corners, and ensured better contact between the CFRP and the concrete substrate.
8. After wrapping, each column was enclosed in polyethylene sheeting, then wrapped with foam mats cut to size. Plywood boards were placed over the mats and secured using two ratchet straps. This configuration maintained consistent pressure against the flat sides of the column and reduced the risk of fabric slippage, particularly along the underside.

9. After 24 hours, the external formwork was removed. The wrapped columns were left to cure for a minimum of seven days under ambient laboratory conditions. The polyethylene was left in place throughout the curing period to protect the CFRP surface from dust and contamination until testing.

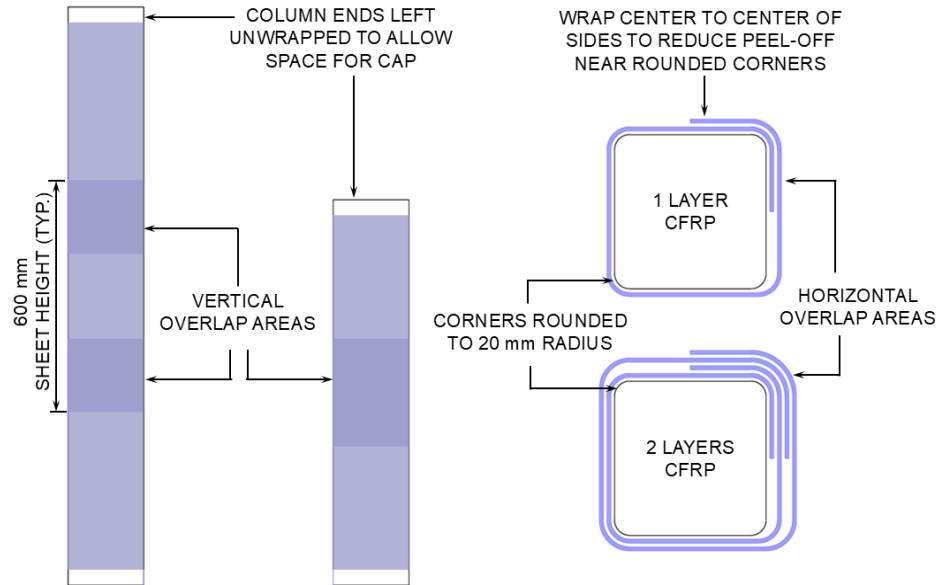
All columns were wrapped horizontally while resting on cinder blocks to ensure safe handling of the heavy specimens during application. This setup also created sufficient clearance to apply CFRP underneath the column, though it introduced time constraints as epoxy began to sag under gravity. It is worth mentioning that the epoxy cures through an internal chemical reaction between its resin and hardener components and does not rely on oxygen or air exposure. Therefore, applying plastic sheeting to the exterior surface of the CFRP wrap during curing does not inhibit the bond development at the CFRP–concrete interface.

3.2.4 Test Setup

Axial compression tests were conducted at the Lakehead University Structures Laboratory using a 500-kip (2224 kN) universal testing machine (UTM). All specimens were tested under pin-ended boundary conditions to simulate realistic axial loading. Each column was capped with steel plates and levelled using quick-set grout to ensure uniform load distribution.

A displacement-controlled loading rate of 0.5 mm/min was applied for all tests, with the exception of the ambient slender column, which was the first specimen to be tested and initially loaded at 1.5 mm/min before the rate was reduced to 0.5 mm/min. Lateral displacements were monitored using two linear variable differential transformers (LVDTs) mounted at mid-height on orthogonal faces. $LVDT_x$ measured displacement along the column's x-axis (width), while $LVDT_y$ recorded displacement along the y-axis (depth). Strain and LVDT data were recorded through a data acquisition system (DAQ). Axial load and vertical displacement were

simultaneously captured using the UTM's internal control system. A representative view of the test setup, including end conditions and instrumentation layout, is provided in Figure 3.3.



(a)



(b)



(c)

Figure 3.2 CFRP retrofitting sequence: (a) Overlap layout and wrap orientation; (b) Corner preparation for fire-exposed columns; (c) Wrapped column with pressure-application setup

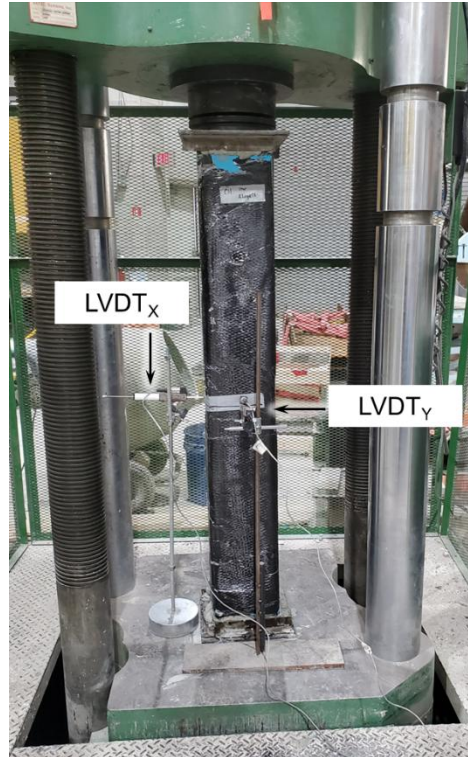


Figure 3.3 Axial compression test setup with pin-ended supports, instrumentation layout, and orthogonal LVDTs for lateral displacement measurement

3.3 Results and Discussion

3.3.1 Load-Normalized Axial Displacement Relationship and Strength Recovery

To establish the effectiveness of CFRP confinement as a post-fire rehabilitation strategy, the experimental axial response is first examined across varying levels of thermal exposure and retrofitting. Key metrics include peak load, load-axial displacement behaviour, and the degree of strength recovery relative to both unwrapped and undamaged specimens. The mechanical behaviour of all tested GFRP-RC columns under axial compression is presented in Figure 3.4. Axial force was recorded directly from the UTM, while axial displacement was obtained from the machine-reported vertical shortening and normalized by the total column height. Column stress is taken as the applied compression force divided by the cross-sectional area. Marker symbols were

added for visual clarity and accessibility. These results offer initial insight into the influence of CFRP confinement, slenderness, and fire exposure on the structural response.

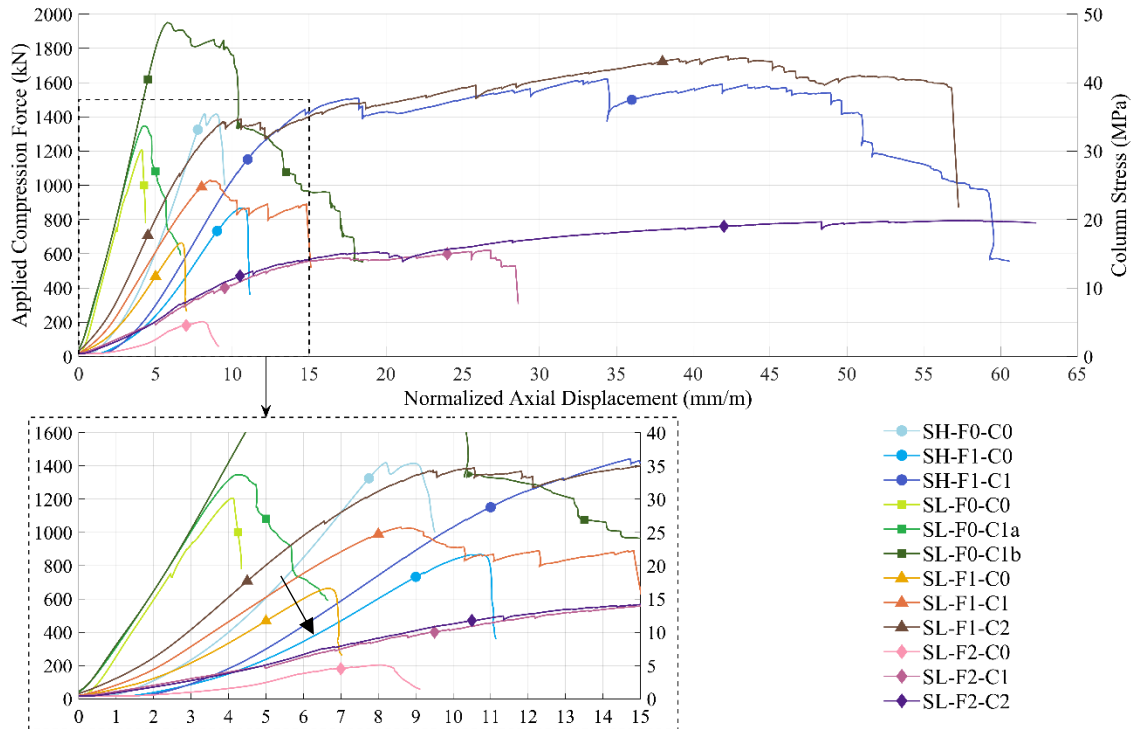


Figure 3.4 Axial load vs normalized axial displacement response of GFRP-RC columns with and without CFRP retrofit; inset highlights early-stage behaviour and confinement effects at low axial displacement

Unwrapped columns exhibited brittle failure with minimal post-peak deformation. In many cases, particularly among wrapped specimens, audible cracking was observed throughout testing and corresponded to sudden drops in axial load, visible on the load-displacement curves. Most wrapped specimens showed two distinct response stages: a steeper initial slope dominated by the unconfined concrete core, followed by an extended deformation phase with varying post-peak response. This trend was especially clear in SL-F1-C2, transitioning near 12 mm/m and continuing beyond 50 mm/m. Similar behaviour was present in other wrapped columns, although not always with as distinctly defined phases. This transition aligns with prior research on FRP-confined concrete, which similarly reported multi-stage axial responses and enhanced deformation capacity due to confinement effects (Mirmiran & Shahaway, 1997; Shayanfar & Barros, 2024). CFRP

confinement effects were clearly reflected in the mechanical behaviour of the three short columns tested. The unwrapped ambient specimen, SH-F0-C0, exhibited a typical brittle response, reaching 1418.7 kN at 8.2 mm/m displacement, with a stiffness of 171.1 kN/mm/m and a maximum displacement of 9.5 mm/m. Fire exposure reduced both stiffness and capacity in SH-F1-C0 (79.5 kN/mm/m, 866.7 kN), though displacement at peak load increased slightly to 10.7 mm/m. By contrast, the wrapped, fire-tested column, SH-F1-C1, exhibited markedly different behaviour. This specimen reached 1621.8 kN at 34.4 mm/m displacement and continued deforming up to 60.6 mm/m, despite its lower stiffness (46.7 kN/mm/m). These results highlight the capacity of CFRP wrapping to alter post-fire response, enabling greater deformation and delaying failure onset. SH-F1-C1's ductile behaviour and sustained load-carrying ability were comparable to those of the slender columns retrofitted with two layers, despite its shorter height and single-layer wrap.

Slender columns showed a wider range of behaviours depending on fire exposure level and retrofitting. The unwrapped ambient control, SL-F0-C0, reached a peak load of 1206.1 kN at 4.1 mm/m displacement, with a stiffness of 289.6 kN/mm/m and a maximum displacement of 5.6 mm/m, indicating a brittle response. Among the specimens wrapped with one layer of CFRP, SL-F0-C1a and SL-F0-C1b achieved increased capacities of 1346.6 and 1950.4 kN, with slightly higher displacement at peak load (4.3 and 5.8 mm/m) and maximum displacement of 6.7 and 18.5 mm/m, respectively. Their stiffness remained high (304.5 and 329.9 kN/mm/m), reflecting minimal confinement activation under ambient conditions.

Fire-exposed, unwrapped specimens SL-F1-C0 and SL-F2-C0 had reduced capacities of 664.0 and 204.1 kN, with corresponding displacement values of 6.7 and 8.0 mm/m and stiffness values of 96.8 and 23.5 kN/mm/m. In contrast, CFRP-retrofitted fire-damaged specimens demonstrated significantly enhanced deformation capacity. Columns wrapped with a single layer,

SL-F1-C1 and SL-F2-C1, reached 1029.8 and 621.4 kN at displacements of 8.6 and 26.7 mm/m, with maximum displacement of 15.1 and 28.6 mm/m, respectively. Specimens with two CFRP layers, SL-F1-C2 and SL-F2-C2, exhibited even greater deformability, achieving 1753.5 and 793.6 kN at 42.3 and 57.6 mm/m displacements, and maximum displacements of 57.0 and 62.3 mm/m. Despite lower stiffness values—ranging from 22.6 to 13.5 kN/mm/m—these specimens maintained ductile behaviour and carried load over extended deformation ranges.

To further evaluate the effectiveness of CFRP confinement, experimental capacities were compared directly to the corresponding unwrapped counterpart to quantify strength gains achieved through retrofitting. The short fire-exposed column SH-F1-C1 reached 187% of its unwrapped counterpart (SH-F1-C0), confirming a substantial benefit even with one CFRP layer. Among ambient specimens, SL-F0-C1a and SL-F0-C1b reached 112% and 162% of SL-F0-C0, respectively. The reduced strength of SL-F0-C1a likely reflects early-age damage and imperfect wrapping, as the specimen fell during handling and represented the first attempt at CFRP confinement, underscoring the sensitivity of confinement performance to application quality and workmanship. For fire-exposed slender columns, strength increased by 155% for SL-F1-C1 (one layer), 264% for SL-F1-C2 (two layers), 304% for SL-F2-C1, and 389% for SL-F2-C2. These improvements underscore the robust confinement effect achieved through external wrapping, even under severe thermal damage.

To assess how the rehabilitated fire-exposed columns performed relative to their original, undamaged condition, each specimen was also compared to its ambient unwrapped reference. SH-F1-C1 reached 114% of the ambient short-column strength (SH-F0-C0), indicating full recovery and modest enhancement. For slender columns exposed to one hour of fire, SL-F1-C1 recovered to 86%, while SL-F1-C2 surpassed ambient strength at 146%. However, after two hours of

exposure, SL-F2-C1 and SL-F2-C2 reached only 52% and 66%, respectively. These results show that while CFRP wrapping can restore, and in some cases improve, original performance, full recovery becomes less achievable as fire duration and thermal degradation increase. Despite this limitation, CFRP confinement remains a practical and effective strategy for post-fire rehabilitation, particularly in cases of moderate thermal exposure.

3.3.2 Comparison to Theoretical Predictions

This section evaluates how well existing axial capacity equations predict the behaviour of GFRP-RC columns subjected to fire exposure and CFRP retrofitting. Specifically, it compares experimental results to theoretical predictions generated using CSA S806-12 (R2021) unconfined and confinement-based expressions, as well as the empirical equations proposed by Shan et al. (2019). All predictions were calculated using ambient-temperature material properties, which do not account for fire-induced degradation in concrete strength, GFRP bond capacity, or CFRP confinement effectiveness. Additionally, both the CSA S806 and Shan et al. models were developed for steel-reinforced concrete (SRC), introducing uncertainty when applied to GFRP-RC systems.

As such, this chapter provides one of the first quantitative assessments of these equations' reliability for fire-damaged, CFRP-retrofitted GFRP-RC columns. The goal is to determine whether current design guidance adequately captures confinement behaviour under post-fire conditions, or whether modifications are needed to improve prediction accuracy.

3.3.2.1 Unwrapped Columns – CSA S806

The theoretical unconfined axial capacity, P_{uc} , was calculated using provisions in CSA S806-12 (R2021) in combination with CSA A23.3-19 that neglects axial resistance from GFRP reinforcement. For short columns ($L=1000$ mm), P_{uc} was taken as the nominal axial capacity P_{ro}

(Equation 3.1), while for slender columns ($L=1500\text{mm}$), a buckling reduction was applied and

$P_{uc} = P_r$ (Equation 3.2), where:

$$P_{ro} = \alpha_1 f'_c (A_g - A_{GFRP}) \quad (3.1)$$

$$P_r = \frac{P_{ro}}{1 + P_{ro}/P_{cr}} \quad (3.2)$$

In these equations, α_1 is the concrete stress block factor, f'_c is the cylinder compressive strength of concrete. The gross cross-sectional area of the column is denoted by A_g , and A_{frp} is the combined cross-sectional area of the four longitudinal GFRP bars. Here, P_r is the slenderness-adjusted axial resistance, and P_{cr} is the Euler-based buckling capacity is used to account for instability in taller columns. Calculations assumed pin-ended conditions (i.e., $K = 1.0$), and the effective length, L_e , was taken as the full column height. This adjustment reflects the increased vulnerability of taller columns to instability effects and enables more realistic comparisons to experimental data.

When compared to experimental results, the CSA unconfined equations produced variable levels of agreement. Among ambient specimens, the unwrapped slender column SL-F0-C0 exceeded its predicted capacity ($P_{ex}/P_{uc} = 106.1\%$), likely due to continued curing at the time of testing. In contrast, the short ambient column SH-F0-C0 reached only 93.2% of its theoretical value. These differences suggest that the unconfined equation may be sensitive to early-age effects and conservatively biased for short specimens. Notably, slender columns retrofitted with one CFRP layer, SL-F0-C1a and SL-F0-C1b, reached 95.2% and 137.8% of their predicted unconfined capacities, respectively. While SL-F0-C1a performed similarly to the unwrapped control, SL-F0-C1b exhibited a substantial strength increase, highlighting confinement effects that are not captured by the CSA expression. These outcomes suggest that even under ambient conditions,

CFRP wrapping can influence axial resistance, particularly when confinement is more effectively engaged.

For fire-exposed, unwrapped columns, P_{ex}/P_{uc} ratios dropped significantly, reaching as low as 19.3% for SL-F2-C0 and 61.4% for SH-F1-C0. This reflects the severity of thermal degradation and highlights the limitation of unconfined predictions in post-fire conditions. The CSA expression offers a useful baseline but does not account for fire-induced reductions in material properties or bond degradation in GFRP bars, both of which contribute to the observed strength losses. In contrast, retrofitted fire-damaged specimens often exceeded their unconfined predictions, with SL-F1-C2 and SL-F2-C2 reaching 165.6% and 75.0%, respectively. SH-F1-C1 (short, one layer) also surpassed its baseline, reaching 114.9%. These results underscore the limitations of the CSA unconfined approach for evaluating retrofitted specimens, where CFRP confinement alters axial resistance, particularly in specimens with multiple layers. In such cases, the CSA model can significantly underpredict capacity, further justifying the need for confinement-specific design expressions when assessing post-fire rehabilitation strategies.

3.3.2.2 *Confined Capacity Using CSA S806*

The CSA S806-12 (R2021) confinement model is evaluated for its ability to predict the axial capacity of CFRP-wrapped concrete columns exposed to fire. The approach assumes linear enhancement of compressive strength based on the lateral confining pressure provided by the external jacket. Using this confining pressure, the axial capacity of retrofitted specimens can be estimated as in Equations 3.3 and 3.4:

$$f_l = \frac{(2n_{CFRP} t_{CFRP} f_{CFRP})}{D} \quad (3.3)$$

$$P_{cc} = (f'_c + k_l k_c f_l) A_g \quad (3.4)$$

Here, the confining pressure f_{lcsa} , calculated using Equation 3.3, depends on the number of layers n_{CFRP} and thickness t_{CFRP} of the CFRP, the section dimension $D = 200$ mm, and the limiting CFRP stress, f_{CFRP} , taken as the lesser of $0.006E_{CFRP}$ or the ultimate tensile stress of the CFRP material, f_{CFRPu} . The confinement effectiveness parameter k_l was calculated as $k_l = 6.7 (k_c f_l)^{-0.17}$, where the confinement coefficient $k_c = 0.4$ for square sections with rounded corners. The confined axial capacity, P_{cc} , was calculated directly using the confinement-adjusted concrete strength, applied over the full cross-sectional area, A_g .

Comparison of CSA-predicted confined capacities (P_{cc}) with experimental results (P_{ex}) revealed consistent overestimation across all retrofitted specimens. Even in the ambient condition, P_{ex} reached only 70.2% of the theoretical prediction for the slender column with one layer of CFRP, suggesting that the current version of CSA S806 may not conservatively capture the confinement response of square columns. The model's assumptions—uniform hoop strain development, ideal bonding, and constant confinement effectiveness do not fully reflect the behaviour of real-world retrofits, particularly for non-circular sections.

For fire-exposed specimens, the gap between P_{ex} and P_{cc} widened, but clear patterns emerged. In the 1-hour group, increasing confinement from one to two CFRP layers raised the P_{ex}/P_{cc} ratio from 46.4% to 62.1%, indicating that while CSA overpredicts, it does qualitatively capture the relative effect of added confinement. However, the magnitude of improvement was significantly less than predicted, likely due to internal degradation that reduced jacket engagement. In contrast, the 2-hour specimens showed virtually no change in the P_{ex}/P_{cc} ratio, 28.0% and 28.1% for one and two layers, respectively, despite an increase in predicted strength. This suggests that beyond a certain damage threshold, the concrete core is too compromised to benefit from additional confinement, an effect not accounted for in the code's linear strength formulation.

These findings also point to a broader limitation: CSA S806 does not differentiate between new and existing columns. In practice, CFRP jackets are rarely applied to undamaged members; however, the model assumes idealized conditions regardless of thermal history or existing damage. The consistent overprediction observed in this study reinforces the need for a more nuanced, damage-sensitive approach. To date, no published studies appear to have evaluated the predictive accuracy of CSA S806 for damaged FRP-retrofitted columns. This absence underscores a broader need to recalibrate confinement models to reflect the performance of rehabilitated members under non-ideal conditions.

3.3.2.3 *Confined Capacity Using Empirical Method (Shan et al., 2019)*

In light of the limitations observed in the CSA S806 predictions, an alternative confinement model was explored to assess whether empirical approaches could better reflect the axial behaviour of CFRP-wrapped square columns. The method adopted here is based on the empirical equations proposed by Wu and Wang (2009) and later calibrated specifically for square columns with rounded corners by Shan et al. (2019). Unlike the CSA approach, which assumes a linear additive strength enhancement from confinement pressure, the empirical model uses a power-law relationship between effective lateral pressure and axial strength gain, derived from regression analysis of experimental results. The equations used in this study reflect Shan et al.'s recommended parameters for square sections and are summarized as seen in Equations 3.5 and 3.6:

$$f_l = \frac{2 \cdot k_e \cdot E_{CFRP} \cdot \varepsilon_{CFRP} \cdot t_{CFRP} \cdot n_{CFRP}}{b} \quad (3.5)$$

$$P_{em} = f_{co} \left[1 + 7.62 \cdot (r_c)^{0.35} \cdot \left(\frac{f_l}{f_{co}} \right)^{1.70} \right] A_g \quad (3.6)$$

Here, f_l is the effective lateral confinement pressure provided by the CFRP, calculated using the CFRP modulus $E_{CFRP} = 115.7$ GPa, design hoop strain $\varepsilon_{CFRP} = 0.0095$, and wrap thickness

$t_{CFRP} = 1.3$ mm per layer, multiplied by the number of CFRP layers applied (n_{CFRP}). The efficiency factor is taken as $k_e = 0.45$, slightly lower than the typical value of 0.50 recommended by Shan et al. (2019) for square columns with rounded corners ($r_c = 0.2$) to account for the relatively thick CFRP fabric used in this study. The column width $b = 200$ mm, and the corner radius effect was incorporated using the normalized corner ratio ($r_c = 2r/b$) with $r = 20$ mm.

P_{ex}/P_{em} ratios ranged from 38.1% to 107.4%, with the highest values observed in slender columns wrapped with two CFRP layers. These results reflect variable agreement between theoretical and experimental values. While some specimens exceeded the predicted capacity, others fell short, particularly after prolonged fire exposure. Among ambient-wrapped specimens, SL-F0-C1b achieved 95.1% of its predicted P_{em} , while SL-F0-C1a reached 65.7%. The short fire-exposed column SH-F1-C1 reached 84.2%, closely aligning with its predicted value. Among slender columns exposed to a 1-hour fire, SL-F1-C1 (one layer) achieved 63.1% while SL-F1-C2 (two layers) exceeded prediction at 107.4%. In contrast, the 2-hour fire specimens performed significantly lower: SL-F2-C1 and SL-F2-C2 reached only 38.1% and 48.6%, respectively. These findings reinforce the trend that confinement effects, and thus strength recovery, become increasingly limited as thermal degradation intensifies, even when nonlinear confinement behaviour is considered.

3.3.2.4 Comparison of Experimental and Theoretical Capacities

To synthesize the findings from the preceding sections, this final comparison evaluates how the three theoretical models, CSA-unconfined (P_{uc}), CSA-confined (P_{cc}), and empirically confined (P_{em}), align with the experimental results across varying levels of fire exposure and CFRP confinement. Table 3.2 summarizes all predicted and measured axial capacities, including the corresponding P_{ex}/P_{uc} , P_{ex}/P_{cc} , and P_{ex}/P_{em} ratios.

As noted earlier, all theoretical predictions rely on material properties measured at ambient temperature and were originally calibrated for undamaged, steel-reinforced concrete columns. These baseline assumptions likely contribute to the consistent overestimation observed in fire-damaged specimens, but the extent of divergence varies by model. By comparing their predictive behaviour in a side-by-side format, this section highlights not only the absolute accuracy of each approach but also its reliability under different retrofit conditions.

Table 3.2 Summary of experimental and theoretical axial capacities for all specimens

Specimen ID	f_{cu}	Experimental Capacity	Unwrapped Theoretical Capacity (CSA S806)		Wrapped Theoretical Capacity (CSA S806)		Wrapped Theoretical Capacity (Shan et al., 2019)	
	(MPa)	P_{ex} (kN)	P_{uc} (kN)	P_{ex}/P_{uc}	P_{cc} (kN)	P_{ex}/P_{cc}	P_{em} (kN)	P_{ex}/P_{em}
SH-F0-C0	50	1418.7	1522.6	93.2%	N/A	N/A	N/A	N/A
SH-F1-C0	46	866.7	1411.6	61.4%	N/A	N/A	N/A	N/A
SH-F1-C1	46	1621.8	1411.6	114.9%	2617.8	62.0%	1927.0	84.2%
SL-F0-C0	39	1206.1	1137.3	106.1%	N/A	N/A	N/A	N/A
SL-F0-C1a	50	1346.6	1415.1	95.2%	2777.8	48.5%	2050.3	65.7%
SL-F0-C1b	50	1950.4	1415.1	137.8%	2777.8	70.2%	2050.3	95.1%
SL-F1-C0	36	664.0	1058.6	62.7%	N/A	N/A	N/A	N/A
SL-F1-C1	36	1029.8	1058.6	97.3%	2217.8	46.4%	1632.6	63.1%
SL-F1-C2	36	1753.5	1058.6	165.6%	2822.6	62.1%	1632.6	107.4%
SL-F2-C0	36	204.1	1058.6	19.3%	N/A	N/A	N/A	N/A
SL-F2-C1	36	621.4	1058.6	58.7%	2217.8	28.0%	1632.6	38.1%
SL-F2-C2	36	793.6	1058.6	75.0%	2822.6	28.1%	1632.6	48.6%

Note: Summary of experimental and theoretical axial capacities. "N/A" indicates specimens not applicable for confinement models due to lack of CFRP wrapping.

The unconfined (P_{uc}) predictions were consistently the most conservative, averaging 1222 kN, with corresponding P_{ex}/P_{uc} ratios ranging from 19.3% to 166.6%. These results provide a useful lower bound but fail to capture confinement effects. The CSA-confined predictions (P_{cc}), on the other hand, were the most optimistic, averaging 2608 kN, with P_{ex}/P_{cc} ratios spanning from 28.0% to 70.2%. This reflects CSA's linear treatment of confinement pressure and high effectiveness assumptions, which become increasingly unconservative in damaged or square-section retrofits. The empirical model predictions (P_{em}) averaged 2029 kN, with P_{ex}/P_{em} ratios from 32.3% to 95.1%, indicating improved conservatism and more stable alignment with experimental data, particularly in moderately damaged specimens.

Although all models predict substantial gains relative to unconfined capacity, the consistent gap between CSA and empirical predictions highlights important design implications. CSA S806 tends to overstate confinement effectiveness in square retrofitted columns, particularly when degradation limits jacket engagement. In contrast, the empirical model, while still imperfect, offers a more conservative and realistic benchmark. This reinforces the importance of model selection in post-fire retrofit design, where assumptions of full confinement may no longer hold. A visual comparison of all axial capacity values is provided in Figure 3.5 to complement the tabulated results.

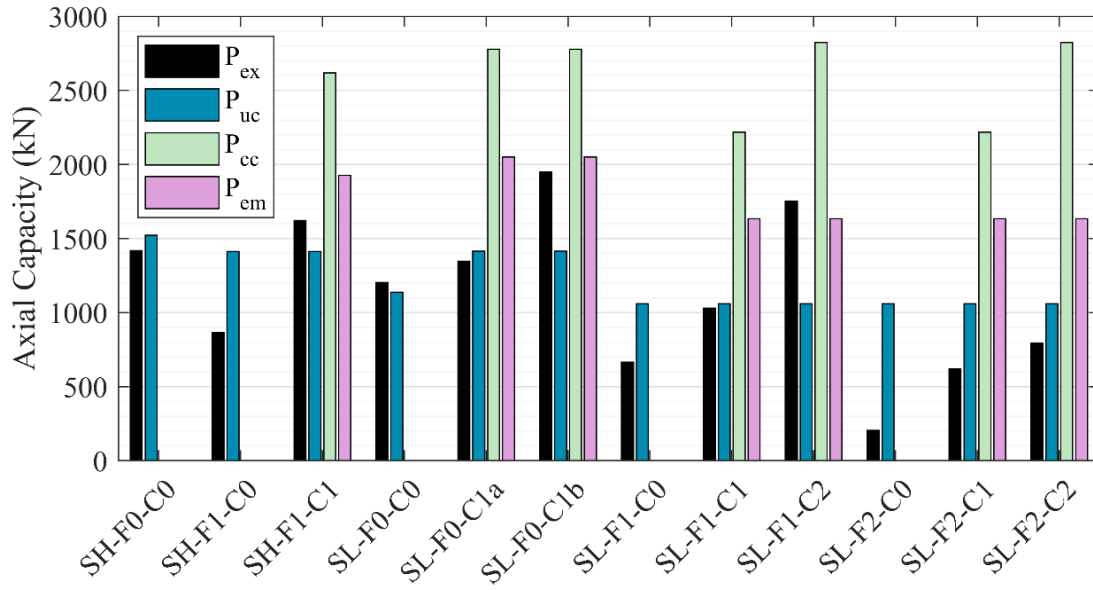


Figure 3.5 Comparison of axial capacities for GFRP-RC columns based on experimental results (P_{ex}), unconfined predictions (P_{uc}), CSA confinement model (P_{cc}), and an empirical model by Shan et al. (P_{em})

3.3.3 Failure Modes and Lateral Deformation and Strain Measurements

Beyond strength enhancement, CFRP retrofitting had a significant impact on how columns responded to axial loading. Retrofitted specimens exhibited markedly different deformation patterns, jacket behaviours, and failure mechanisms compared to their unwrapped counterparts. While responses varied depending on fire exposure, slenderness, and retrofit configuration, several clear trends were observed across the test matrix, particularly in relation to out-of-plane instability and progressive jacket deterioration.

Lateral displacement data from Figure 3.6 reveal pronounced out-of-plane displacements in several fire-exposed slender columns, particularly in specimens wrapped with two CFRP layers. SL-F2-C2 recorded the highest y-direction drift (62.7 mm), including 47 mm of additional displacement measured beyond the LVDT range, visually confirmed by the inset in Figure 3.7b, while SL-F2-C1 exhibited the greatest x-direction displacement (18.4 mm). Although SL-F1-C2 was also equipped with LVDTs, its mounting stands became twisted during testing, rendering the

data unreliable. However, photographic evidence (Figure 3.7a) clearly illustrates substantial x-direction bending in SL-F1-C2, and approximate scaling from these images suggests displacements on the order of ($x \approx 72$ mm, $y \approx 20$ mm). While these values should be interpreted cautiously, they support the conclusion that SL-F1-C2 experienced deformation comparable to SL-F2-C2's y-axis response. Other slender columns, such as SL-F2-C0 and SL-F1-C1, also experienced notable drift. In contrast, unwrapped specimens (e.g., SH-F0-C0 and SL-F0-C0) remained near the origin with total displacements below 5 mm. These trends suggest that fire exposure and slenderness had a more significant impact on lateral deformation than CFRP confinement level alone.

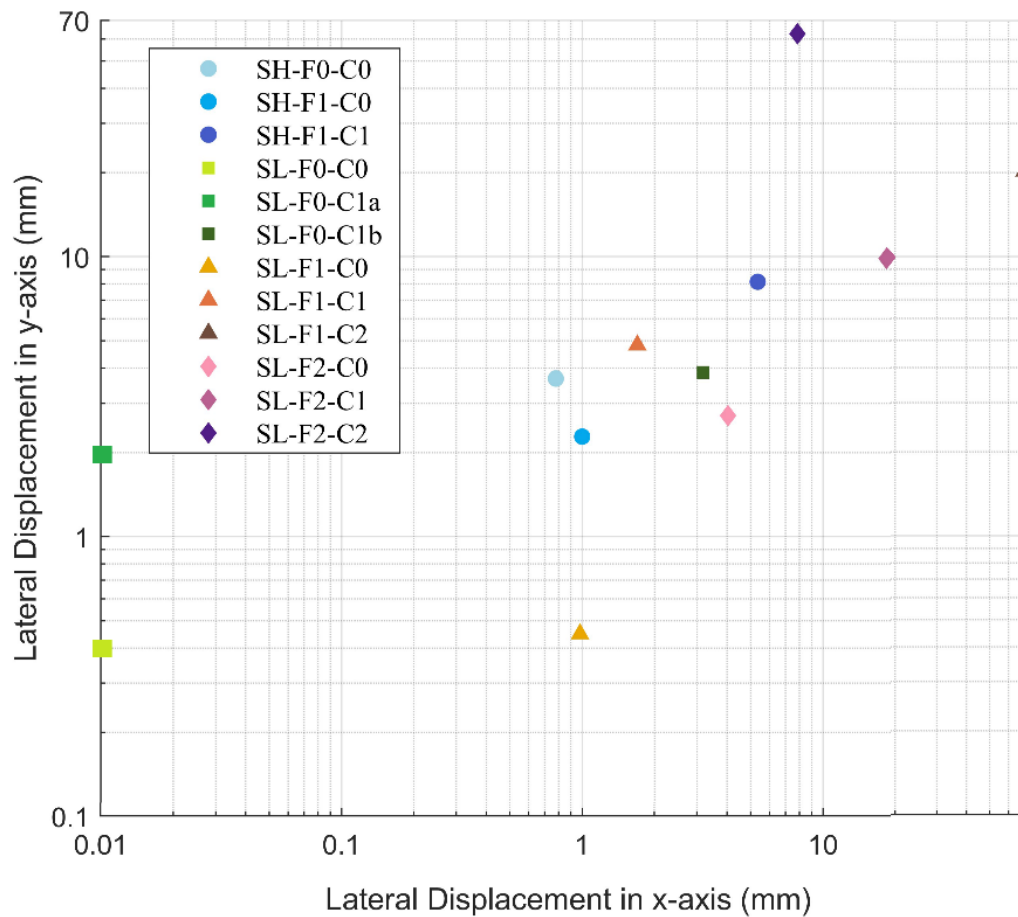


Figure 3.6 Mid-height lateral displacements of GFRP-RC columns at peak load in x- and y-directions

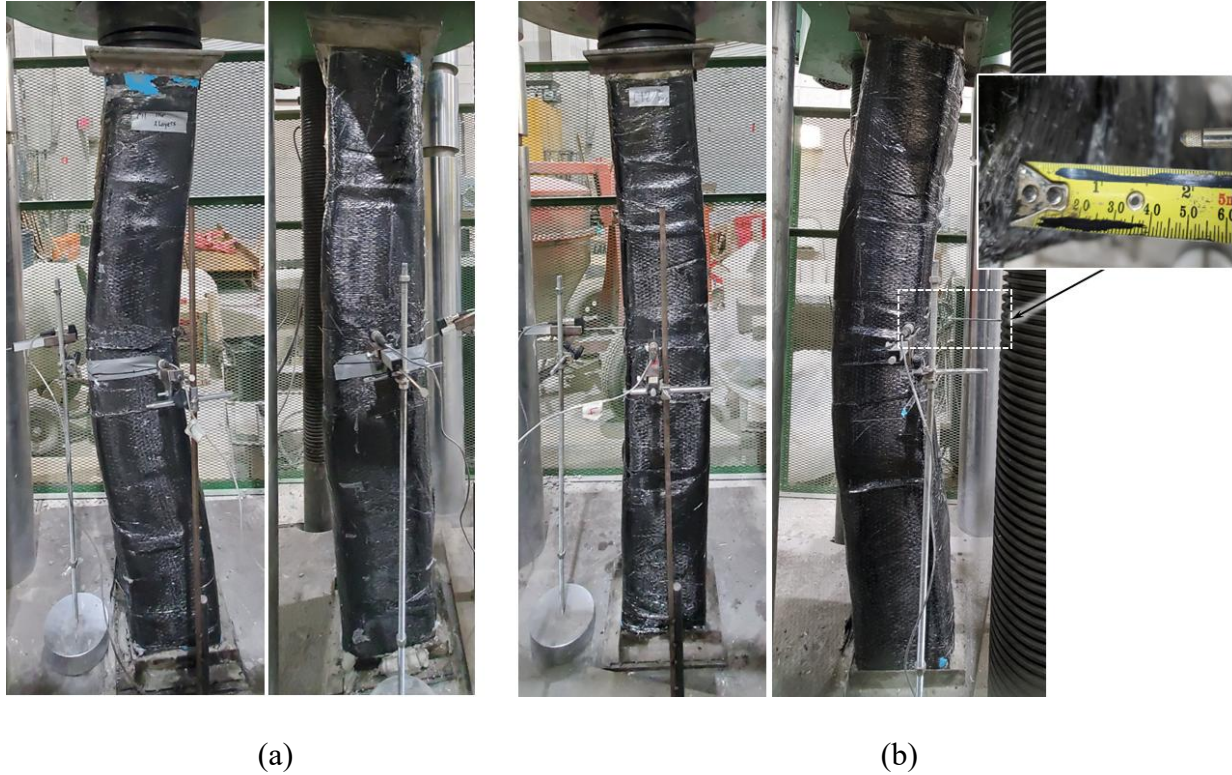


Figure 3.7 Out-of-plane deformation in slender retrofitted columns, shown in x- and y-directions (left and right, respectively), for: (a) SL-F1-C2; (b) SL-F2-C2, including additional 47 mm displacement in y-direction measured beyond LVDT range

CFRP failure characteristics from across the test matrix are illustrated in Figure 3.8 using SH-F1-C1 as a representative example. This specimen displayed multiple localized jacket defects observed in other retrofitted columns, including mid-height buckling, lower-third distortion, and seam bulging. These features, visible to varying degrees in other specimens, reflect common patterns of CFRP degradation and help contextualize the failure modes discussed below. Wrapped columns generally exhibited two distinct CFRP failure modes: either delamination between CFRP layers, where the jacket detached in large sheets, or progressive shredding and rupture into smaller fragments. The former was seen in specimens like SL-F0-C1a and SL-F1-C1, while the latter occurred in SH-F1-C1, SL-F0-C1b, and SL-F2-C1. SL-F1-C2 exhibited an intermediate failure pattern—its jacket did not fully delaminate but also did not shred; instead, large torn segments separated irregularly, suggesting partial interlayer bonding. SL-F2-C2, by contrast, deformed

continuously without visible jacket rupture and was removed from the UTM with its CFRP still intact. An effort was made to identify patterns in the failure mode of the CFRP by examining maximum load, strength relative to unwrapped and ambient specimens, axial and orthogonal displacements, and test duration; however, no consistent trend emerged to distinguish between delamination and fibre shredding.

Strain gauges were limited to unexposed specimens, as fire exposure would degrade the sensors (the sensors adopted in this study are not resistant). SL-F0-C1a was excluded due to previously noted fabrication issues. Among the three tested specimens, as expected for the unwrapped specimens, the short specimen (SH-F0-C0) reached a higher strain compared to the corresponding slender column (SL-F0-C0) at the level of maximum/peak load for each specimen, where the strains were recorded as 1600 micro-strain and 1420 micro-strain, respectively. On the other hand, the specimen with wrapping, but not exposed to fire (SL-F0-C1b), exhibited the highest strain of about 2260 micro-strain. It is worth mentioning that the SL-F0-C0 exhibited the most uniform strain response across all four GFRP bars, with minimal variation between sensors. In contrast, SH-F0-C0 and SL-F0-C1b showed wider disparities in individual gauge readings. SL-F0-C1b demonstrated the greatest overall strain engagement, with three of its gauges exceeding 1800 micro-strain. These differences reflect the localized nature of deformation and cracking within each column.

These outcomes, including both the observed failure patterns and internal strain measurements, highlight the role of epoxy condition, wrap continuity, and confinement effectiveness in shaping how CFRP fails. Overall, CFRP jacketing transformed failure from abrupt brittle collapse to a more gradual and observable process. Even when failure did occur, the jacket delayed instability and preserved structural continuity longer, offering potential warning signs.

These findings confirm that confinement not only delayed global instability but also enabled higher internal deformation prior to failure. While a single CFRP layer often improved performance, two layers provided superior post-fire ductility, especially in slender, fire-damaged specimens.

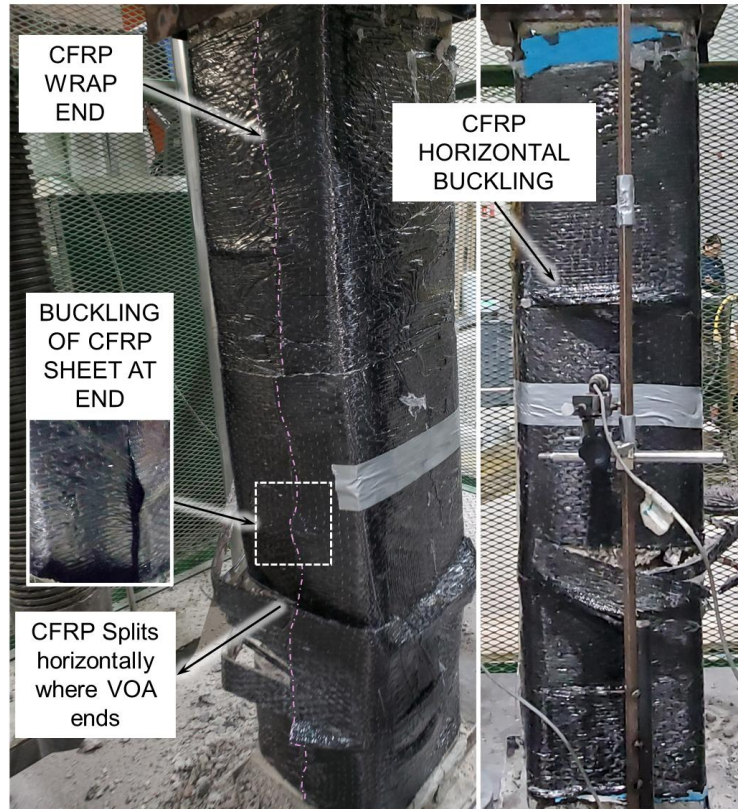


Figure 3.8 Representative CFRP failure modes observed across retrofitted specimens, illustrated using SH-F1-C1. VOA refers to the vertical overlap area between adjacent CFRP wraps.

3.4 Conclusion

This study evaluated the axial performance of GFRP-RC columns retrofitted with externally bonded CFRP jackets following fire exposure. Based on experimental testing of twelve columns with varying fire durations, slenderness levels, and retrofit configurations, the following conclusions can be drawn:

1. CFRP confinement enhanced deformation capacity and shifted failure modes in all retrofitted specimens. While unwrapped columns failed abruptly with limited post-peak

normalized axial displacement, wrapped columns exhibited multi-stage responses, including extended deformation under sustained or gradually declining loads. Maximum normalized axial displacements ranged from 10 to over 65 mm/m, confirming the role of CFRP in improving ductility and transitioning behaviour from brittle to more ductile failure, particularly after fire exposure.

2. Compared to their unwrapped fire-exposed counterparts, retrofitted columns exhibited progressively greater strength gains as fire duration increased. Strength increased modestly under ambient conditions (139%), more significantly after one hour of fire (155–264%), and most dramatically after two hours (304–389%). These trends suggest that CFRP confinement becomes increasingly effective as core degradation intensifies.
3. Despite increased confinement effectiveness with fire exposure, severely damaged columns remained the weakest relative to undamaged ambient references. After one hour of fire, strength recovery ranged from 73% to 125%, depending on CFRP thickness. However, even with two layers, columns exposed to two hours of fire recovered only 44% and 57%, indicating that CFRP alone cannot fully compensate for severe thermal degradation.
4. The CSA S806-12 (2021) confinement model consistently overpredicted axial capacities for CFRP-retrofitted specimens, particularly under post-fire conditions. While it qualitatively captured the influence of CFRP thickness, its assumptions of uniform confinement and ideal bonding led to non-conservative predictions, with experimental-to-predicted strength ratios dropping as low as 28% in fire-damaged columns. These findings underscore the limitations of code-based confinement models in post-damage applications and motivate the use of empirical alternatives calibrated for non-ideal conditions.

5. The empirical confinement model proposed by Shan et al. (2019) provided more conservative and accurate predictions than CSA S806 for square columns with rounded corners. Agreement with experimental data was reasonable in ambient and moderately damaged specimens, but declined substantially under severe thermal exposure. Neither model incorporated degradation of the concrete core or reinforcement, limiting their predictive reliability for post-fire rehabilitation scenarios.
6. CFRP jacketing significantly altered failure modes and deformation behaviour, reducing brittleness and enabling more controlled post-peak response. Lateral instability was most pronounced in slender, fire-exposed specimens, while jacket failures were shaped by confinement quality, epoxy condition, and wrap configuration. Collectively, these results confirm the dual role of CFRP in delaying instability and enhancing ductility, particularly when applied in multiple layers.
7. Rebar strain measurements from ambient-tested columns revealed maximum bar strains ranging from 1500 to 2700 micro-strain at peak load. Strain responses were generally consistent among gauges in SL-F0-C0, while SH-F0-C0 and SL-F0-C1b exhibited greater variability. These results highlight localized bar engagement and offer a baseline for interpreting deformation behaviour in the absence of internal data for fire-exposed specimens.

Chapter 4 - Conclusions and Figure Work

4.1 General Findings

This thesis investigated the behaviour of GFRP reinforced concrete columns under fire exposure and evaluated the effectiveness of CFRP jacketing as a post-fire rehabilitation strategy. The goal was to address key knowledge gaps related to the thermal vulnerability of FRP materials and the limited guidance available for post-fire assessment and strengthening of non-metallic reinforced compression members. This was accomplished through a two-phase experimental program involving fourteen large-scale GFRP-RC columns exposed to standardized furnace conditions and later evaluated under axial compression.

In the first phase of testing, fire exposure was found to significantly reduce axial capacity, with strength losses increasing alongside fire duration and column slenderness. The short fire-exposed column retained 61% of its ambient counterpart's strength, while slender columns dropped to 55% and 17% after one- and two-hour exposures, respectively. Slender specimens also experienced faster internal heating, more pronounced lateral deformations, and a higher tendency toward instability, confirming that increased surface exposure amplifies both thermal and structural degradation. Thermocouple data showed that critical thermal thresholds were exceeded in multiple regions, contributing to resin loss, fibre splitting, and delamination. Failure in fire-damaged columns was abrupt and brittle, with minimal post-peak deformation and visible spalling or buckling.

Differences in axial load behaviour and reinforcement performance were also observed. One specimen tested under sustained axial load during fire retained higher residual strength than its unloaded counterpart (65% vs 55%) but exhibited lower ductility, suggesting that load presence may have influenced cracking patterns and damage evolution. This result implied that the residual

capacity observed in the other, unloaded fire-exposed columns likely represents a conservative lower bound. The ambient-tested unreinforced column reached 83% of the strength of its GFRP-reinforced counterpart, indicating that the GFRP bars contributed minimally to axial capacity, primarily affecting stiffness or confinement interaction rather than direct capacity. These observations support current design provisions, such as CSA S806, which conservatively neglect GFRP contributions in fire-exposed compression members.

The second phase investigated post-fire rehabilitation, where seven specimens were retrofitted with CFRP wraps. CFRP jacketing improved both axial capacity and deformation behaviour in fire-damaged GFRP-RC columns. Retrofitted specimens exhibited more ductile responses with delayed failure, increased peak strain, and multi-stage load–strain behaviour. While one layer of CFRP contributed between 150% and 200% strength recovery for ambient and one-hour fire-exposed specimens, the addition of a second layer increased strength to over 250% for SL-F1-C2. Two-hour fire-exposed specimens benefited even more: SL-F2-C1 reached over 300%, and SL-F2-C2 nearly 390% of their unwrapped counterparts. However, when compared to ambient references, only SH-F1-C1 and SL-F1-C2 managed to regain full or greater capacity, highlighting the challenges of confinement-based retrofitting under severe thermal degradation.

Code-based confinement models were found to overestimate strength in many retrofitted specimens. The CSA S806 approach, while capturing general trends with increasing CFRP thickness, assumed ideal confinement and bonding conditions that did not reflect the experimental results. The empirical model by Shan et al. (2019) provided more conservative predictions, particularly for moderately damaged columns, but also showed limitations under severe fire exposure. These findings indicate that current confinement models require calibration for post-fire applications and square-section geometry.

4.2 Research Contributions

This thesis contributes to the advancement of knowledge in the area of fire performance and rehabilitation of non-metallic reinforced concrete components. The main research contributions are summarized as follows:

1. Conduct large-scale fire testing of GFRP-RC columns under realistic conditions:

Perform large-scale fire tests on GFRP-reinforced concrete columns using CAN/ULC-S101 standard fire exposure. The test matrix incorporates variations in slenderness, fire duration, and sustained axial loading to assess axial capacity loss and failure behaviour.

2. Assess theoretical axial capacity for fire-damaged GFRP-RC columns using CSA S806:

Apply CSA S806-12 (R2021) provisions to estimate the axial capacity of unwrapped, fire-exposed GFRP-RC columns. Predictions aligned well with experimental results for ambient specimens when GFRP contribution was neglected, consistent with current design assumptions. However, fire-damaged columns exhibited significantly lower capacity than predicted, highlighting the need for fire-specific reduction factors in future design provisions.

3. Investigate CFRP jacketing for post-fire rehabilitation of GFRP-RC columns:

Study the use of externally bonded CFRP wraps to improve strength and stability in fire-damaged GFRP-RC columns. Results show that confinement enhances axial capacity and deformation response across a range of damage levels and column geometries.

4. Evaluate theoretical confinement models in fire-damaged columns:

Compare axial capacity predictions from CSA S806-12 (R2021) and an existing empirical model against test results for CFRP-confined columns. Discrepancies observed under post-fire conditions

emphasize the need for improved confinement design guidance, particularly for non-ideal configurations.

4.3 Recommendation for Future Work

This study contributes to addressing key gaps related to the fire performance and rehabilitation of GFRP-reinforced concrete columns. However, several areas remain critical before this understanding can be confidently implemented in practice. The following recommendations aim to improve predictive models, refine confinement strategies, and support the broader adoption of FRP-based components in fire-prone or remote environments:

1. **Propose design-oriented fire reduction factors for unwrapped GFRP-RC columns:**

Fire exposure caused substantial strength losses in unwrapped GFRP-RC columns. Future research should develop and validate simplified reduction factors for use in design standards, enabling more accurate estimation of residual axial capacity after fire.

2. **Examine one-sided fire exposure and asymmetric thermal gradients:** The present study considered four-sided heating only. Future research should investigate columns subjected to non-uniform fire exposure, where differential thermal expansion and degradation can cause bowing toward the heated face. Understanding whether GFRP reinforcement can contribute to restraining or exacerbating such behaviour is critical for extending fire design guidance to realistic compartment fire conditions.

3. **Investigate the influence of tie configuration and spacing on residual behaviour:**

Increasing transverse reinforcement may improve confinement and reduce instability in fire-damaged GFRP-RC columns, consistent with observations from ambient-condition studies on FRP-reinforced columns. Future work should assess whether similar benefits

extend to post-fire conditions and whether greater tie density should be reflected in design provisions for fire-resilient columns.

4. **Calibrate confinement models for post-fire CFRP-retrofitted columns:** Existing confinement models (e.g., CSA S806, Shan et al. 2019) overestimated strength in several retrofitted specimens. Additional testing is needed to refine model parameters for square sections, degraded cores, and imperfect confinement conditions.
5. **Evaluate the long-term durability of CFRP-retrofitted columns:** The current study focused on short-term axial loading. Follow-up research should consider environmental exposures such as freeze–thaw, moisture ingress, UV radiation, and aging to ensure continued performance after fire.
6. **Incorporate realistic mechanical loading, including eccentric or cyclic conditions:** All columns in this study were tested under concentric, monotonic loading. Introducing eccentricity or cyclic loads would improve understanding of confinement behaviour under realistic post-fire demand scenarios.

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